

An Advanced Course in Nuclear Engineering

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Nuclear Structural Engineering



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An Advanced Course in Nuclear Engineering

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This nuclear engineering textbook series aims to serve as global education standard for all universities that have a department of nuclear engineering and all research laboratories and companies related to nuclear engineering.

As global warming issues are strongly being advocated, nuclear power is examined again as a low-carbon and ecology-friendly energy source. Furthermore the accident at TEPCO Fukushima Daiichi Nuclear Power Plant on March 11, 2011, has intensified the attention to nuclear engineering. Industrial countries with nuclear power are not confined to domestic sales but are working toward global sales strategies of nuclear power plants, involving many nations. Against such a background, while nuclear power industry develops globally, the required human resources education should keep up. However, no English textbook with the potential to become the global standard has been published so far, although awaited for a long time. Recognizing the demand, Japanese nuclear power specialists concentrated their expertise and a Japanese nuclear engineering textbook series was published under the cooperation of the University of Tokyo, the Japan Atomic Energy Agency and other organizations related to nuclear engineering. The needs of such comprehensive textbooks are growing remarkably after the accident. Japan has steadily advanced the research and development of all fields of nuclear engineering. This is reflected in the broad coverage of the series, which includes not only nuclear reactors, but also the nuclear fuel cycle, radiation use, maintenance, etc. This English version of the Japanese series serves domestic as well as foreign institutions. Although the Japanese version of the series was published before the accident, readers will find the in-depth coverage of diverse topics in nuclear engineering still relevant.

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Nuclear Structural Engineering

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Preface

Nuclear reactor structural engineering, a basic subject in nuclear engineering, is the study of the structural integrity of the nuclear reactor, which anyone who designs, constructs, operates, or maintains nuclear reactors must understand.

This textbook is designed to help develop the general understanding of all the knowledge essential to ensuring the reactor's structural integrity. For this purpose, it not only explains basic knowledge of nuclear reactor structural engineering but also allows readers to learn how to systematize this knowledge as they study structural standards.

As is well known, in March 2011, a large-scale earthquake occurred and a huge tsunami triggered by this earthquake struck the eastern coast of the main island of Japan. The tsunami caused a severe accident at Units No. 1 to No. 3 of the Fukushima Daiichi nuclear power plant (Daiichi: literally, No. 1), resulting in release of a large amount of radioactive materials to the environment. (Hereinafter, this severe accident is referred to as the “Fukushima Daiichi nuclear accident,” and the earthquake as the “2011 Tohoku Oki Earthquake.” (Tohoku: literally, northeastern region of the main island of Japan, and Tohoku Oki: off the coast of Tohoku region)). We, as members engaged in nuclear industry, are unbearable to shame about the severe accident. The accident makes us keenly conscious of self-questioning at any time “what ‘ensuring nuclear safety’ is” and “how structural integrity is assured.” In reflecting lessons learned from the accident, Japanese regulatory organizations discussed critical issues on safety regulations and then have drastically strengthened technical issues such as safety design and countermeasures against natural hazards as well as regulatory frameworks. Safety improvements have been unremittingly pursued by the nuclear industry as a whole with nuclear reactor licensees and regulatory authorities as leading organizations. As part of the safety improvements, an Integrated Regulatory Review Service (IRRS) was conducted by the International Atomic Energy Agency (IAEA) at the request of the Government of Japan. As a result, further regulatory reforms including inspection system improvements have been performed for clarifying the responsibility for ensuring nuclear safety and for promoting voluntary measures for enhancing safety by the licensees. Accordingly, significance of studying

how structural standards are upgraded by incorporating unremittingly reforming safety regulations and the latest knowledge and findings is growing.

From this perspective, the textbook consists of chapters focusing on the following topics.

Chapter 1 is an introductory part that describes the structure of the book and refers to the relations between chapters. Strength of materials, structural mechanics, the finite element method, and structural integrity constitute the knowledge particularly important to nuclear reactor engineers in studying structural integrity. Among them, this chapter focuses on structural standards, which are considered to be relatively unfamiliar to readers, and explains their concept. The chapter also mentions trends in safety regulations that everyone dealing with structural standards as a nuclear reactor engineer should know, as well as current and future developments expected in relation to structural standards. Especially, how and why the safety regulations have been strengthened are summarized from aspects of nuclear reactor structural engineering, referring to lessons learned from the Fukushima Daiichi nuclear accident and so forth.

Chapter 2, which is aimed at those who have learned strength of materials in engineering courses at universities, explains the basics of strength of materials, which is the knowledge requisite for chief engineers of reactors. These days, most calculations for the design of structural strength, depend on commercially available numerical analysis codes that use the finite element method. However, without basic knowledge on strength of materials, it is impossible to define a problem (plane stress, plane strain, beam, buckling accompanied by large deformation, etc.) and generate appropriate input data (load application, boundary conditions, material properties, etc.). This knowledge is also indispensable for understanding calculation results. As a means to verify the results, comparison of theoretical expressions and values in ideal cases is performed as presented in Chap. 2. Where these are not available, an experiment is conducted to compare and analyze results. As such, knowledge of strength of materials is necessary for structural strength calculations even when computer simulations are performed. This knowledge is also essential to professionals working at nuclear power plants. For technicians, it is important to have the capability to manually and instantly calculate rough estimates of stress, strain, and deformation due to a rise in internal pressure, load, or temperature within a pressure vessel or piping.

In Chap. 3, readers will learn the basics of material strength and structural strength that are prerequisite for understanding failure modes assumed in structural standards. This will help them get ready to study structural design methods from Chap. 5 onward. With respect to materials, Chap. 3 provides a general explanation of elastoplasticity, creep, and other properties that invalidate the linear relationship between stress and strain hypothesized in Chap. 2, along with failure modes related to these properties. The chapter also describes diverse failure modes of structures that depend on not only material characteristics but also the nature of shape and load.

Chapter 4 focuses on the finite element method as a means to predict stress and strain generated in component structures involving complexity in shape or boundary conditions. Theoretical solutions learned through strength of materials are not enough to calculate such stress and strain. After explaining the basic theory of the finite

element method, the chapter outlines the method of verifying modeling methods and solutions required for analyzing nuclear reactor structures of high reliability.

Chapter 5 explains assumed failure modes and concepts for preventing each failure mode, which both constitute knowledge that should be understood about the structural design of the light water reactor (LWR) in order to ensure the structural integrity of LWR components. Because of the need to use terms unfamiliar to most readers, the explanation is provided step by step, from basic concepts to practice. We expect that readers will learn the concepts of ensuring structural integrity, or in other words, how constituent technologies are systematically organized as structural standards. The chapter also pays attention to the historical development of these concepts. In addition, trends in structural design standards are outlined, including design methods using the finite element method. As response to the Fukushima Daiichi nuclear accident, improvements in concepts of ensuring structural integrity are introduced, with focusing on those reflecting thorough implementation of the defense-in-depth approach for assuring nuclear safety and measures against a severe accident.

Chapter 6 describes the structural design method for fast reactors that operate in the high-temperature range where creep deformation occurs to materials. The structural design of fast reactors must take into account creep-related failure modes, in addition to failure modes assumed in LWRs. Moreover, a wider variation in coolant temperature due to higher operating temperature creates severer thermal stress. The chapter provides an overview of the characteristics of the structural design of fast reactors based on specifications different from those of LWRs, the design procedure that takes account of these characteristics, and structural standards for fast reactors.

Chapter 7 introduces, first, the so-called “Seismic Design Review Guide” formulated in 2006 and explains its background, underlying ideas, and points to keep in mind, while taking a look at the historical development of seismic design concepts. Next, the tsunami-resistant design and the relevant issues of seismic design that have been drastically changed from the above Review Guide are explained, referring to failure in assuming the huge tsunami generated by the 2011 Tohoku Oki Earthquake and insufficient tsunami countermeasures taken at the Fukushima Daiichi nuclear power station. The chapter particularly focuses on reasons for and concepts of the classification of facilities according to their seismic importance, ideas behind the assumption of earthquake ground motion according to seismic importance, the method of calculating earthquake ground motion, and the concept of allowable limits for evaluating seismic safety. Finally, the chapter mentions basic issues in practical applications.

Chapter 8 summarizes how components and structures are fabricated following the structural design procedures explained in the previous chapters. The chapter describes methods to ensure high quality in nuclear reactor facilities, including materials manufacturing technologies and heat treatment. A description of welding, an essential element of component manufacturing, is also provided, including the outline of its standards and technologies.

Chapter 9 describes nondestructive examination methods and pressure and leak testing that are used for verifying that the fabricated components and structures meet the design requirements and are of appropriate quality. Considering that the method

chosen for nondestructive examination should be optimal in light of the form of the defect to be examined, the chapter explains what kinds of defects can occur, with attention to their relations with failure modes. Based on this concept, this chapter explains which inspection method is appropriate for which type of defect, along with the characteristics of each inspection method. Further, describe the improvements in inspection systems such as weld examinations/inspections that were regulated in 2020 on the basis of recommendations by the IAEA IRRS mentioned above.

In Chap. 10, readers will learn about the latest overview on fracture mechanics, which is the knowledge needed for evaluating integrity in cases involving the application of standards assuming the existence of cracks—designed for safety-critical components in order to ensure higher structural reliability—and cases where any cracks have been detected in components during examination. This is an important concept because components are not free of the possibility of the occurrence and development of cracks that may have an impact on structural integrity even when the quality and integrity of the components are ensured by applying strict conditions for materials selection and by performing construction and in-service inspections. This possibility is caused by aging phenomena, such as fatigue and stress corrosion cracking, which occur as components are used for a long time period after the start of operation.

Chapter 11 first describes technical standards concerning the maintenance of integrity as well as standards for integrity maintenance, as part of a system of standards for evaluating component structural integrity using fracture mechanics. Then, the chapter explains methods to evaluate crack development and procedures for integrity evaluation as examples of integrity evaluation techniques meeting specification standards.

In closing, we hope that readers will read through this and other textbooks on nuclear energy and become competent nuclear reactor engineers who can make decisions on various issues facing nuclear energy from a comprehensive point of view and can lead the world with an advanced problem-solving capability.

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Message from the Series Editors

Since the beginning of the twenty-first century, expectations have been rising in Japan and abroad on nuclear power generation because the need for safe and stable energy supply is increasing against the background of global environmental issues and the depletion of oil-based energy sources. The situation is calling for the development of human resources with advanced knowledge and techniques of nuclear energy. However, many traditional textbooks on nuclear engineering have fallen out of print, and many of them were written about 30 years ago when full-scale R&D and use of nuclear energy began in Japan and therefore hardly reflect recent advances.

Anyone who tries to solve and counter practical problems and unknown application problems encountered in using nuclear energy will face the need to return to the basics. This requires the capability to envision various images of the subject. The systematic acquisition of knowledge of nuclear energy is fundamental to nuclear R&D and the development of the nuclear industry. This series of textbooks is designed to serve this purpose.

With advances in the utilization of nuclear energy, there have emerged many challenges that cannot be solved by the traditional nuclear engineering approach, which addresses the relationship between society and nuclear energy in an individualized and fragmented manner. Traditional education that uses scientific and technological approaches is not enough to help understand and handle these problems appropriately. In an effort to reduce risks and uncertainties associated with the use of nuclear energy, the University of Tokyo has started the world's first systematic education and research that address the humanities and sociological aspects of nuclear energy. Some of the textbooks in this series are used in these courses.

We have published this series as a set of standard textbooks for systematically studying nuclear energy from basics to practice, with the aim of nurturing experts and engineers who can act from a global perspective. While being aware of the situation surrounding nuclear energy, students need to not only familiarize themselves with basic knowledge but also acquire practical expertise, including flexible knowledge and first-hand techniques that allow them to understand field practices without feeling any gap.

This series consists of three courses on nuclear energy: the basic course, the frontier course, and the practical course. The content is based on the education at the Nuclear Professional School and the Department of Nuclear Engineering and Management, both established by the University of Tokyo with the aim of developing high-level human resources specializing in nuclear energy to cope with the new century of nuclear energy. The textbooks were written mainly by faculty members of the University of Tokyo and researchers at the Japan Atomic Energy Agency and in related industries.

We would like to extend gratitude to all those who have kindly taken time to contribute to or cooperate in the creation and publication of this textbook series.

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Chapter 1

Introduction



When a rod is stretched from both ends, it deforms until it breaks, or ruptures, to be precise. The situation is different in the case of components like a heat exchanger. For example, when internal pressure is applied, a heat exchanger tube expands and, before breaking like a rod, may deviate from the configuration that is necessary for the achievement of the required performance. Once deformed like this, the component can no longer provide the required performance. The state of being unable to fulfill the required performance due to fracture, distortion, and so forth is referred to as the loss of structural integrity. To prevent loss of integrity, how should you fabricate and use components? How can you demonstrate that the given component maintains integrity? This book explains how you can fabricate components in a way that ensures integrity and how you can see whether the integrity of the component is maintained, while referring to the latest concepts.

The basic principle is to consider the uses of the component before choosing materials for it and fabricating it so that its integrity is assured while in use. The key is how to decide the configuration and dimensions of the component before fabrication. This involves setting the allowable range where the integrity of the component is assured under the loads expected during use and defining the configuration and dimensions so that the component will be kept within the allowable range. This is the most important element in designing the component. Then, the fabricated component is subjected to examination in order to verify that it has been fabricated according to the design and provides integrity. Although for the sake of integrity, fabrication should take place in a way that prevents harmful “flaws,” it is also critical to ensure that even somewhat large flaws will not damage integrity. “Structural standards” are descriptions of the requirements to be met in order to ensure the integrity of components, including materials selection, design, fabrication, and examination.

This textbook first defines structural integrity and explains the concept of structural standards that is used for ensuring integrity, and then describes strength of materials (or mechanics of materials), which is vital as a means of designing components’ dimensions and other elements, and the finite element analysis, which has

been developed on the basis of strength of materials and has been going mainstream in recent years. Subsequently, the textbook explains how components are designed, fabricated, and examined in practice. In particular, with respect to design, the book places emphasis on what should be considered in sizing of components and provides a description of practical activities of sizing and other elements. It is difficult to perfectly remove a harmful flaw even though an examination is properly implemented. Therefore, this book introduces how to assess the integrity of structure involving a harmful flaw.

1.1 Structure of the Textbook

This chapter, the introduction to the whole book, briefly explains the role that structural standards play in ensuring safety, the most important factor to nuclear power facilities, and then describes the concepts underlying the current structural standards. Subsequently, the chapter describes how structural standards have evolved along with technology development, and their directions of development as a concept, while referring to the trajectory of related safety regulations. Regarding safety regulations, it is explained how the regulations have been changed as the response to lessons learned from the severe accident at the Fukushima Daiichi (literally, Fukushima No.1) nuclear power station that is owned by the Tokyo Electric Power Co., Inc. (TEPCO). Currently, the Tokyo Electric Power Company Holdings occurred in March 2011. Hereinafter, the accident is referred to as the Fukushima Daiichi nuclear accident. Then, how structural standards have been modified is described.

Chapter 2 explains strength of materials, which is a fundamental and critical tool to predict stress and deformation caused by the application of a load to components. This prediction is essential to the sizing of components. A slightly detailed description is provided on the energy principles, which serve as the base for structural analysis by the finite element method, an analysis method important in practical work. Chapter 3 describes how materials and structures are deformed and fractured under a load, for the purpose of deepening your knowledge of strength of materials. This helps you to understand phenomena associated with deformation and failure expected to occur to components. Chapter 4 explains structural analysis based on the finite element method, which is commonly used in practical design processes, with emphasis on application.

Based on the above knowledge, the essence of structural standards is introduced in Chap. 5 and thereafter. Chapters 5 and 6 explain design in relation to the concepts of how to ensure structural integrity against loads (e.g., internal pressure, thermal load, dead weight) at events caused by the plant's internal factors during normal operations, an accident, and so on. Chapter 5 focuses on structural design concept in relatively low temperature ranges where the creep phenomenon hardly occurs, as in the LWR. Particular emphasis is placed on what kinds of fractures and failure are expected to occur to components, and how to make sizing of components in order to prevent such fracture and failure (the distinction between these two are described

in the same chapter) under the applied load. Chapter 6 explains high-temperature structural design, or design in high-temperature ranges where the creep effect is pronounced, as in the fast reactor. At high temperature, the load resulting from temperature differences, known as the thermal load, is important (the thermal load is called the thermal stress when it is regarded as a stress). For this reason, the chapter mentions not only the failure prevention but also how to reduce the thermal load. Relations between force or the load and stress are described in Sect. 1.4, Chap. 2.

Chapter 7 highlights the concepts of structural design against the earthquake load, or the concept of seismic design. Unlike incidents arising from internal factors mentioned above, earthquakes cannot be controlled by humans and are therefore categorized as an externally caused incident. Since preventing earthquakes is impossible, they should be addressed by approaches different from those used for internally caused incidents. This is why a chapter separates from Chaps. 5 and 6 is dedicated to this topic. With regard to seismic design, a major revision was made to the Regulatory Guide for Reviewing Seismic Design of Nuclear Power Facilities in 2006, based on experiences of large-scale earthquakes like the 1995 Southern Hyogo-ken (literally, Hyogo Prefecture) Earthquake and achievements of R&Ds so far. Furthermore, tsunami-resistant design concepts were drastically changed reflecting lessons learned from the Fukushima Daiichi nuclear accident, at which the facilities were mainly damaged by the tsunami generated by the 2011 off the Pacific coast of Tohoku Earthquake (hereinafter referred to as the 2011 Tohoku Oki (literally, off the coast of Tohoku) Earthquake). Since design methodologies for earthquakes and tsunamis have been practically developed on a basis of experiences and lessons learned from large-scale earthquakes and strong tsunamis, Chap. 7 explains concepts of seismic design and tsunami-resistant design, referring to a historical background of both design methodologies.

Chapter 8 explains the fabrication of components, while mentioning material selection and production, although these are not in order of the construction process. Chapter 9 outlines examinations aimed at proving the quality of fabricated components, with emphasis on the defect detection methods.

Chapter 10 introduces you to fracture mechanics, the study of the handling of cracks, focusing on up-to-date concepts, and considering that the theories of strength of materials cannot be applied to components with cracks. Chapter 11 addresses structural integrity evaluation techniques that use fracture mechanics, constituting key elements on how to assess the integrity of components with cracks, with concepts of the prevention of brittle fracture.

1.2 Nuclear Safety and Structural Standards

Nuclear energy plays an essential role in power supply in Japan as well as Europe and America. With rising concerns over the depletion of oil resources, nuclear energy is attracting attention, especially in Asian countries. This is particularly true of China, where the construction of nuclear power plants is forged ahead with as a vital measure

to meet soaring demand for energy. Lessons learned from severe accidents at the Three Mile Island Nuclear Power Plant (NPP) unit No.2 (TMI-2) that occurred in 1979 and at the Chernobyl NPP unit No.4 in 1986 strengthened greatly the relevant safety measures. Because of the strong awareness of the importance of measures to combat global warming since the beginning of the twenty-first century, high expectations are placed on the revival of nuclear power generation. At the same time, hydrogen production using high-temperature gas-cooled reactors as the heat source has begun to regain attention, following steady advances in the High-Temperature engineering Test Reactor (HTTR) in Japan.

However, the Fukushima Daiichi nuclear accident happened in March 2011. This severe accident led to significant improvements of safety measures not only in Japan but also worldwide. Further, the regulatory regime in Japan was drastically changed.

Due to large amounts of radioactivity contained in the reactor core, nuclear power plants involve a potential risk of radioactive release to the environment. Strictly ensuring safety is critical at these facilities in order to prevent this from materializing, or in other words, to protect the health and safety of the public and site personnel from radiation and radioactive risks potentially arising from plant operations or an accident there. For this reason, nuclear power plants are required to promise high quality levels that allow them to meet their performance and functional requirements by a sufficient margin. However, since the functionality needed for ensuring safety varies from structure to structure, from system to system, and from component to component, at the nuclear power plant, it is necessary to target rational and appropriate quality levels according to the relative importance of each component.

A key concept for the safety assurance of nuclear power plants is defense-in-depth. This intends to ensure safety mainly through five hierarchical safety measures, namely, the defense-in-depth concept is a graded approach of safety assurance, which is structured in the five levels. Although we do the best efforts to maintain the effectiveness of the first-level safety measures, in a case when the first level (or line) of safety measures (or safety activities, defense) fails, and the protective barrier for this level is threatened, the subsequent second level will be activated and functioned to ensure safety. Thus, successively if the fourth level fails, the fifth or last level comes to play. The detail can be explained as follows:

- First-level safety measures: These are aimed at preventing defects from occurring to operating systems and components necessary for safety assurance and avoiding the occurrence of abnormal incidents (anticipated operational occurrences) due to trouble, operating errors, etc.
- Second-level safety measures: Even if first-level safety measures have been taken, it is possible that defects occur to components or that abnormal incidents occur and escalate due to operating errors. They can also occur if the measures taken have been insufficient for some reason. The second-level safety measures are aimed at preventing the situation like this from escalating into an accident. The plant should be brought back to normal operating conditions as soon as possible [1].

- Third-level safety measures: Even if an accident occurs, these measures are taken to stop it from escalating, to control the accident within design basis ranges, to mitigate its consequences, and to prevent abnormal radioactive releases. It is required to return the reactor to a safe state. Although very unlikely, the accidents are postulated to occur in the safety design. Design basis accidents (DBAs) are a set of accidents that are used to define the design bases in the design. The objective of considering the DBAs is that the safety measures are designed on the DBAs encompassing the limiting loads of sets of similar events, i.e., representative of groups of events with similar plant behaviors, with regard to each of safety design criteria (for example, overpressure criterion), because it is not practical to consider case-by-case events.
- Fourth-level safety measures: Reflecting lessons learned from the aforementioned severe accidents, even if the third level safety measures were to fail, the subsequent level of safety measures should be available to prevent the progression of the accidents that results from such failure and to mitigate the consequence in order to keep the release of radioactive materials as low as reasonably achievable. Such severe accidents include those that may be caused by multiple failures. Severe accidents are defined such accidents to develop significant damage to a reactor core, or to fuel assemblies or spent fuels that are stored in a nuclear fuel material storage equipment. Severe accidents, etc., involve accidents that have a fear to reach the severe accident conditions (excluding anticipated operational occurrences and design basis accidents. This exclusion means that severe accidents, etc., are categorized as so-called beyond-design-basis accidents.) in addition to severe accidents. Referring to the defense-in-depth concept by the International Atomic Energy Agency (IAEA), the objective of the fourth-level safety measures (corresponding to the fourth level of the IAEA's concept, which is also structured in five levels) is to control the severe plant conditions even in such severe conditions [3].
- Fifth-level safety measures (the last level as defense-in-depth): Although the first four levels of safety measures are effective in limiting the consequence of a severe accident, significant release of radioactive materials is assumed as the fifth level, and measures are taken to mitigate the radiological consequence. Off-site emergency plans as well as on-site ones should be prepared. In the Japanese regulatory framework, the Act on Regulation of Nuclear Source Material, Nuclear Fuel Material, and Nuclear Reactors (hereinafter, referred to as the "Reactor Regulation Act") addresses the first four levels of safety measures, while the Basic Act on Disaster Management and the Act on Special Measures Concerning Nuclear Emergency Preparedness (hereinafter, referred to as the "Nuclear Emergency Act") address the last fifth level of safety measures in order to take measures for protecting public's lives, health, and property, preserving the environment and so forth, because the fifth level closely concerns the nuclear disaster outside the plant site. It should be noted here that an abbreviation of direct translation into English about the Reactor Regulation Act may be the "Reactors, etc. Regulation Act." However, use of the Reactor Regulation Act is prevalent among Japanese regulatory bodies and licensees, especially owners of electric power companies.

Taking into account of such situation, the Reactor Regulation Act is used as the abbreviation.

Actions to be taken as plant engineering for the first four levels of safety measures are as follows.

The first-level safety measures involve selecting appropriate materials, designing, fabricating and examining components with advanced technologies, with sufficient margins, and conducting appropriate quality assurance activities, in order to protect components from failure and trouble. In addition, highly reliable operating procedures and maintenance are essential.

The second-level safety measures should prevent, in the event of an abnormal incident, the incident from expanding and escalating into an accident. This involves installing safety protection systems and safety equipment. In the case of components, this means taking design-oriented measures that stop an abnormal incident from expanding and escalating into an accident.

The third-level safety measures include engineered safety features such as containment vessel and emergency core cooling system as provisions for the occurrence of an accident in order to prevent abnormal release of radioactive materials to the environment and to return the reactor to a safe state. Safety design requires to prevent secondary accidents. For example, this means a design to prevent a pipe rupture accident from causing rupture in other pipes. In such a case, the design of safety equipment should take into account, as a design requirement, that a load generated by a pipe rupture should not cause rupture in other pipes. The above safety design requirement imposes the structural design requirement that other components than the ruptured pipe should assure structural integrity at the accident and, therefore, that a structural standard should address integrity assessment in such accident conditions.

The fourth-level safety measures may involve additions of equipment or require extension of capacities of equipment that are installed as the first three levels safety measures so as to maintain the integrity of a containment vessel at severe accidents, etc. (as mentioned above, severe accidents, etc., involve not only severe accidents but also beyond-design-basis accidents with possibilities of escalating to severe accidents).

The Reactor Regulation Act in Japan addresses the first four levels of safety measures: First, to require measures to sufficiently cope with design basis events in order to control the plant within the design basis conditions and to return to a safe state at the end of the event, and second, even if the first three levels safety measures fail and then, severe accidents, etc. occur, to require the fourth-level safety measures so as to prevent severe accident progression or to mitigate severe accident consequence resulted from release of radioactive materials (fission products) outside the plant site. Accordingly, nuclear reactor facilities are classified into the following two classes [4].

- (a) Facilities prepared as the first three levels safety measures: To include facilities for preventing undue external release of radioactive materials and for returning the reactor to a safe state in operating conditions from normal operations to design basis accidents. These facilities are designated as “facilities for design

basis events” (or, design basis event measures facilities), because these facilities are required to function at design basis events including design basis accidents, which define the safety design bases.

- (b) Facilities prepared as the fourth-level safety measures: These are designated as “facilities for coping with severe accidents, etc.” (or, “severe accident measures facilities”). Extensions of capacities of safety systems belonging to facilities for design basis events may be required for functioning at severe accidents, etc. Such safety systems should be classified into the class of facilities for coping with severe accidents, etc., while they belong to facilities for design basis events.

Regarding rules that structural standards stipulate for meeting the aforementioned safety requirements, in short, structure, systems, and components of a nuclear power plant are required to perform safety functions (functions required for safety assurance) not just during normal operations but even in the event of an abnormal incident (anticipated operational occurrence, to be more exact), a design basis accident and a severe accident. A level of quality that should be ensured for the purpose of achieving safety functions depends on the assumed situation—normal operations, an anticipated operational occurrence, a design basis accident or a severe accident. The quality level depends also on the importance of component. This has created the concept of classification of operating conditions, or classification of service conditions. Further, the concept of component classification has been developed, and then, rules on design, fabrication, examination, operation, and so forth are provided for ensuring the different quality level depending upon the component class. Accordingly, structural standards define requirements for guaranteeing that each component maintains sufficient quality level, with no fracture or degradation in performance and functionality, in any of the above four situations, that is to say, not just during normal operations but also in the event of an accident, and in accordance with the importance of component. From a structural engineering viewpoint, structural standards specify what kinds of materials to select, how to design, fabricate, and examine equipment, and even how to operate and maintain it. They further specify qualification of engineers or personnel who are doing quality assurance activities as well as the above-mentioned technological actions. As mentioned later, structural standards can be called structural technical standards when attention is paid to technological aspects.

1.3 The Goal of Structural Standards

1.3.1 *Target*

Structural standards specify general requirements for maintaining the integrity of machinery and equipment. The goal of structural standards is to balance (or harmonize) the safety and economy of machinery and equipment, which are two seemingly mutually contradictory requirements [5]. Too much emphasis on safety will result in

too costly machinery and equipment that are bulky and cranky. As long as machinery and equipment are used for economic activities, it is a matter of course that not only appropriate economical efficiency but also competitiveness is ensured for them.

In the narrow sense, safety means securing the safety of people in general, including the public and the workers, and preventing environmental contamination. In other words, it means protecting people's health and property, and preserving environment from industrial disasters potentially caused by machinery and equipment. However, safety must be considered from a broader perspective, a perspective that encompasses national safety, and the assurance and improvement of industrial vitality and technical levels [5]. We should be fully aware that industrial disasters not only do harm to people's health and property and cause environmental contamination, but weaken industrial vitality and lower the technical levels of the nation. As demonstrated by the Chernobyl accident, disasters at nuclear power plants can have a huge impact on not just the country concerned but even on neighboring countries. Therefore, nuclear power plants call for consideration of global-scale safety, in addition to domestic safety.

On the other hand, economical efficiency should be improved by proactively adopting the latest technologies, knowledge, and intelligence.

If structural standards fail to adopt up-to-date technologies and knowledge, machinery and equipment that are designed, fabricated, and operated according to those standards will be inferior in economy and fall out of date. Furthermore, such a situation is also undesirable from the viewpoint of rationally pursuing safety. In the midst of fierce international competition, it is of critical importance that structural standards actively adopt cutting-edge technologies and knowledge. This policy is also crucial to the safety of Japan.

1.3.2 Basic Concepts of Structural Standards and Their Overview

Preventing machinery and equipment from fracture and failure by a rational margin is referred to as ensuring structural integrity. In other words, this means assuring rational quality. The basic concepts underlying this are described below.

- (1) First, from the technological viewpoint, L , the force or load (or stress) applied to machinery and equipment, should not exceed R , the resistance (or strength) up to which the machinery and equipment can withstand without fracture or failure. However, there is variation (uncertainty) in the resistance or strength of machinery and equipment as well as in the load to be applied. Consideration should also be given to the potential case where actual resistance is lower than the assumed one due to design error or other reasons. With these taken into account, the load to be applied must be below the resistance by a sufficient margin, DF . Therefore, to balance safety and economy, a rational DF value needs to be set so as to satisfy the following equation:

$$L \leq \frac{R}{DF} \quad (1.1)$$

Since R/DF indicates the upper limit of the permissible load (note that stress is expressed as the permissible load per unit area), it is called the allowable load (or allowable stress, in terms of stress).

Resistance or strength varies depending on factors such as the type and production method of materials constituting the machinery and equipment and the quality of the fabrication method, including the welding method. In addition, the nature and size of detectable defects depend on the kinds of examinations to be performed to detect the potential defects in materials and on the skills of the examiner. Resistance or strength is reliant on the defect remaining in the materials. Although a defect remains existing no matter what kind of examination method is applied, the size of defect to be detected varies depending on the examination method to be applied. Therefore, the size of defect that remains existing after examination is highly dependent upon the examination method. Accordingly, the resistance or strength of equipment that has passed examination can vary depending on the quality of examination. This means that sizing wall thickness and configuration in such a way as to ensure that the applied load (or stress) will not exceed the allowable load (or allowable stress) is not enough to rationally prevent fracture and failure. It is crucial to select appropriate materials that meet the requirements and appropriately produce them (material selection and production), appropriately fabricate the equipment from those materials (fabrication), and perform appropriate examination and testing during fabrication in order to check that those materials and equipment cause no problems (examination, testing, and inspection). These processes help assure proper quality that meets the required performance and functionality, that is to say, assure the required quality level. As a result, appropriate structural integrity is assured.

- (2) Efforts to assuring the required quality level are not limited to construction processes such as material selection and production, design, fabrication, and examination. Even a product most carefully fabricated can deteriorate in quality if used sloppily. Quality assurance efforts should encompass all the processes of construction and use, including the utilization (operation) phase that follows the start of use.
- (3) What is important in assuring quality throughout the process, and by extension in assuring structural integrity, is not just meeting technical requirements (e.g., required wall thickness) concerning construction (fabrication) and utilization (operation), but human factors should also be taken into consideration. More specifically, human factors refer to (1) what kinds of activities are conducted by operators and organizations engaged in each process in order to assure quality (collectively called quality assurance activities), and what kind of system is used for these activities; (2) whether engineers with appropriate skills and competence are involved in each phase of these activities, including design and fabrication phases; and (3) whether the organization is structured to allow this (e.g., welders and examiners with appropriate skills conforming to technical requirements are performing welding and nondestructive examinations). This calls for

the specification of, for example, the required skill levels and ways to guarantee the possession of the required skills (an act known as certification and accreditation). Furthermore, whether ingenuity is encouraged at the workplace is an important factor in creating better quality.

In other words, structural technical standards that specify technical requirements concerning materials, fabrication, examination, maintenance, and so on—namely material standards, design standards, fabrication standards, examination standards, and maintenance standards—are designed to assure structural integrity. Therefore, they ensure that equipment **can** be constructed and operated safely, but **not** that equipment **will** be constructed and operated safely. In order to ensure that equipment **will** be constructed and operated safely, it is essential to adhere to not only appropriate technical requirements and standards but also standards concerning human factors in fulfilling these technical requirements. The latter standards include those for quality assurance, and certification and accreditation. From this perspective, structural standards are systematized based on the hierarchical structure concept as shown in Fig. 1.1.

- (4) Listed below are basic principles for assuring quality, or assuring structural integrity of machinery and equipment, applied to each phase of construction, under the basic concepts described in above (1)–(3).

(i) Material selection/production and fabrication phase

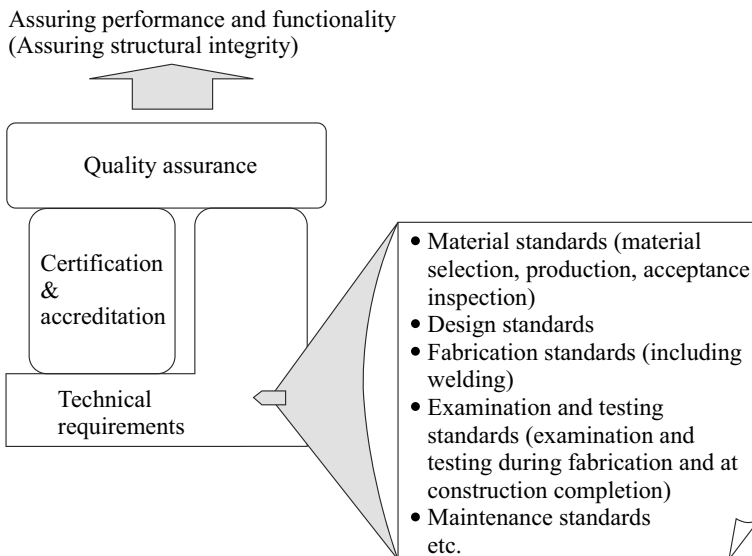


Fig. 1.1 Hierarchical structure of structural standards regarding technical requirements and human-factor requirements (certification and accreditation as well as quality assurance)

- Materials that are capable of achieving the required performance and functionality appropriately should be selected, and they should be manufactured appropriately in accordance with the regulations or specifications. At the stage of fabricating equipment by processing these materials, a fabrication method that is reliable enough to meet the equipment's performance requirements should be used, based on past experiences and other information. Processes conducted at this stage, such as heat treatment, should not damage the specified properties of the materials. In particular, there should be no harmful defects in the materials, including welds (note that in the nuclear power engineering field, damage tolerance design, a design method assuming the presence of defects, is not currently used, in principle). Different people often derive different images from the term "defect." Since whether the defect is harmful or not is critical in structural standards, related terms are strictly defined from this viewpoint. Chapter 9 shows these definitions.
- Materials, including welds, should provide sufficient toughness and ductility. The toughness and ductility levels should be high enough to prevent unstable fracture due to potentially assumed defects. Unstable fracture is a phenomenon where once a defect begins to grow, it rapidly develops under unstable conditions and leads to a fracture.

(ii) Design phase

- Design should ensure that, provided that the absence of harmful defects at the commencement of use is ensured through the above phase and the examination phase below, fracture or failure that may occur during utilization is prevented by a reasonable margin against the load assumed for design (known as the design factor). In other words, wall thickness and structural configuration should be determined in such a way that prevents stress caused to equipment by the load assumed for design from exceeding the reasonably defined allowable stress. The design factor, which is explained in detail in Chap. 5, was formerly called the safety margin or the safety factor. At present, its value is determined deterministically from available data and other information.
- Stresses developed in equipment have been calculated by means of structural mechanics as explained in Chap. 2. These days, they are calculated by using computer codes incorporating stress analysis methods (FEMs) introduced in Chaps. 4 and 6. However, current structural standards provide structural mechanics-based stress assessment frameworks, and therefore, FEM results may be required to elaborate for conforming to the stress assessment procedures. The details are described in Chap. 5.

(iii) Examination phase

- To ensure that all harmful defects have been removed, examination should be performed as necessary, using reliable nondestructive examination methods suited for the properties of the assumed defect.

- To check the pressure resistance and leak tightness of the equipment, pressure testing and leakage testing should be performed. In structural standards, examination is distinguished from testing. The details are provided in Chap. 9.

On the other hand, the principles applicable to the utilization phase are as follows:

- Pressure testing and leakage testing should be performed periodically to assure that structural integrity, including pressure resistance and leak tightness, is maintained.
 - In the construction phase, the equipment is designed so that the required functions are maintained during the service period assumed for design (described in the design specification), without the need for particular measures taken after the commencement of use. However, if a certain kind of aging degradation that was not considered for design occurs, a harmful defect may be caused. To prevent such a defect from triggering fracture, components should be periodically examined according to their significance. If any defect is detected, its growth should be evaluated and repairs or replacements should be made as needed.
- (5) It is desirable that these principles for assuring structural integrity be made rational in consideration of the strictness of the required safety, or more generally speaking, the required quality level. A typical approach to this is defining multiple levels of quality and optimizing the combination of technologies to be applied in each process so that the required quality level is satisfied (the technologies to be combined can usually be chosen from multiple options).

Examples of quality level classification

- Component classification, classification of importance of safety function, and classification of importance in the seismic design at nuclear facilities
- Welded joint classification for pressure-retaining components

As explained in detail in Chap. 5 regarding the construction phase, in the current structural standards, components are classified according to the order of importance. The technologies to be combined (design technology, welding technology, examination technology, etc.) in order to meet the required quality level are fixed for each component classification category. On the other hand, in the utilization phase, examination technologies and acceptance standards for defects are chosen according to the significance of the degradation mechanism and the resulting failure mode.

In closing the section, here is its summary. As Fig. 1.1 shows, structural standards with respect to technical requirements are broadly divided into material standards, design standards, fabrication standards, examination and testing standards, and maintenance standards, according to the processes in the construction and utilization phases. To make it easier to understand the overall picture of the structural standards, the correspondence of the standards and major processes (i.e., which standard refers to which processes) is provided below in order of process. Processes included in

square brackets are existing processes for which no structural standards have been established to date. For such processes outside the scope of the structural standards, standards are responsibly developed by the plant owner and presented as a specification or as standards separate from the structural standards. The parenthesized text indicates in which standards the item is specified. The list includes institutional aspects related to human factors, or worker management aspects.

(a) Technical aspects

- Material selection (material standards).
- Design (design standards).
- Material production and procurement, material acceptance examination (material standards).
- Fabrication (processing, welding, and joining) (fabrication standards).
- Examination (e.g., nondestructive examination of welding) and testing during fabrication and at construction completion (examination and testing standards).
- [Transport and packaging].
- Installation (fabrication standards, for processing and welding for connecting different components; examination and testing standards, for examination and testing at installation).
- [Trial operation].
- [Operation].
- Maintenance and inspection (e.g., examination and testing related to maintenance).
- Examination after the commencement of operation, and repair and replacement in response to detected defects and anomalies (maintenance standards).
- [Dismantling].

(b) Worker management aspects (human factors).

- Quality assurance (quality assurance standards).
- Certification and accreditation (referred to as certification and accreditation standards in this book).

1.4 Technology Innovation and Structural Standards

To keep safety at its best and maintain economic advantages, it is essential to actively introduce the latest technologies. Along with this, structural standards should evolve by incorporating the latest technologies in order to keep the best state. Before describing how structural standards are expected to advance in future, this section touches on several technologies and explains how they have developed and how structural standards have steadily evolved with these developments while embracing them. The structural standards were not as they look like today at first. You must understand that, as explained further in Chap. 5, structural standards have been

constantly evolving since their first appearance in the 1910s in order to prevent recurrence of accidents that were frequently occurred so far, by incorporating up-to-date technologies. They must remain dynamic by embracing the latest technologies.

Technologies for material production, design, fabrication, examination, and so on are steadily advancing every day. For example, design technologies have evolved like this. Until the 1980s, when computational science began to attract attention, equipment had been designed to satisfy the allowable values by conservatively calculating the expected stress and deformation through hand calculations based on strength of materials, which is described in Chap. 2. Wall thickness calculations and stress evaluations were developed so as to suit calculation methods by the strength of materials. The fact that the “design by analysis” method was published in the 1960s, as mentioned in Chap. 5, indicates that even design by analysis is built on the strength of materials. Therefore, complex three-dimensional configurations, such as those of vessel nozzles, were simplified to conservative two-dimensional configurations to which strength of materials’ methods was applicable. However, in the 1980s, analysis techniques based on the finite element method, explained in Chap. 4, went into actual use, making it possible to easily calculate stress and deformation in complex-configuration units and dramatically improving calculation accuracy. This led to improved rationality and reliability of stress calculation. In response to this advance in calculation technology, methods to evaluate stress by directly using the results of finite element analysis were proposed for structural integrity assessment and are starting to incorporate into design standards.

Another example here is concerned with examination technology, namely, nondestructive examination technology to detect defects in structural materials without destruction. When the LWR was introduced to Japan, the only reliable way of detecting internal defects in its structural materials was to use X-ray or γ -ray radiographic examination. However, technology innovation in the years that followed has made such defects detectable with sufficient accuracy by ultrasonic examination, an examination method that involves no radiation exposure. Moreover, technological development has allowed examination results to be kept as records. These advances led to the decision in the late 1990s by the American Society of Mechanical Engineers (ASME) to revise its Boiler and Pressure Vessel Code so as to permit the use of either radiographic examination or ultrasonic examination for the detection of internal defects (but with some exceptions).

Meanwhile, facilities where extraordinary high level of safety is required, such as nuclear power plants, must be certainly protected against catastrophic brittle fracture. Brittle fracture has become widely recognized since failures of cargo ships during the World War II. The method used in those days to prevent brittle fracture was to measure the energy absorbed by a notched test specimen until it fractured and to permit the use of the material only above the temperature at which the material was considered to remain ductile enough. Preferably, this should have been defined by the fracture load (or fracture strength, which is expressed as the fracture load per unit area), but there was no established technique to evaluate these values in those days. Subsequently, as is introduced in Chaps. 10 and 11, fracture mechanics has made significant advances, leading to the development of fracture mechanics

parameters for specifying fracture conditions, such as stress intensity factor, and allowing evaluations of fracture possibility. Today, fracture is rationally prevented by conducting safety assessment of cracked structures based on the stress intensity factor.

This way, structural standards must steadily develop so that they permit the application of cutting-edge technologies in areas like the above. In Japan, as mentioned in Sect. 1.6.1 scientific societies and industrial associations, such as the Japan Society of Mechanical Engineers and the Japan Electric Association, act as private-sector standards' development bodies and revise standards in a timely manner, thus ensuring that their standards embrace the latest technologies.

1.5 The Present and Future of Safety Regulations

To understand structural standards, you must know the concepts underlying safety regulations as a precondition for them. In particular, if you are to understand how structural standards have developed and where they are heading for, you should get a grasp of the current state of safety regulations and the direction of their potential development. This section describes how safety regulations are evolving.

1.5.1 *Structure of Safety Regulations*

Considering the risk involved in nuclear energy, the strictest regulations are imposed on the installer (or owner) of nuclear reactor facilities. The administrative authorities apply the following philosophies in enforcing nuclear safety regulations [6].

- (i) The prime responsibility lies with the owner (or so-called business operator in nuclear power plants, or licensee), and the government shall regulate its business activities by public mandate.
- (ii) Safety regulations shall be effective, reflecting the latest technological findings.
- (iii) Safety regulations shall be clear (or transparent) and available (or open) to the public.
- (iv) Safety regulations shall respond to international trends independently.

In Japan, under the Atomic Energy Basic Law (or Atomic Energy Basic Act (Act No. 186 of 1955)), there are three basic laws to ensure public nuclear safety, which are the basis for the above safety regulations.

- Act on the Regulation of Nuclear Sources Material, Nuclear Fuel Material and Reactors (Act No. 166 of 1957) (as mentioned earlier, referred to as the “Reactor Regulation Act”).
- Act on Special Measures Concerning Nuclear Emergency Preparedness (Act No. 156 of 1999) (hereinafter, referred to as “Nuclear Emergency Act”).

- Act on the Regulation of Radioisotopes, etc. (Act No. 167 of 1957) (formerly, Act on the Prevention of Radiation Hazards due to Radioisotopes, etc.) (hereinafter, referred to as the “Radioisotopes Regulation Act” or “RI Act”).

The above acts have several ordinances and guides as the lower regulations. Their structures are complex. If they are referred to by their respective reference number in this book, it is not easy to distinguish the ordinance or guide. Hence, the ordinances and guides are referred to by their abbreviations or their full names. Further, note that in Japan, nuclear power reactors are regulated also by the Electricity Business Act, because they are also electric facilities for power generation.

The Reactor Regulation Act relates to the installation and operation of nuclear reactors and so on and, then, stipulates rules that the reactor installer must observe at different stages, from installation to operation and decommissioning. Table 1.1 lists framework of the Reactor Regulation Act, focusing on research and test reactors and power generation reactors. This table shows the legal structure of the act amended in 2020, namely, the revised version reflecting not only lessons learned from the Fukushima Daiichi nuclear accident but also results of the Integrated Regulatory Review Service (IRRS) mission conducted by the International Atomic Energy Agency.

Chapter 4 of the Reactor Regulation Act provides rules on installation, operation, etc., of nuclear reactors. According to the amendment of 2020, the Nuclear Regulation Authority (NRA) permits the installation of reactor after reviewing its basic design and then approves the detailed design and construction work plan. Material production and component fabrication start, followed by pre-service examination by licensee. After nuclear regulatory inspection that is carried out by the NRA is passed, the licensee can start the operation of the nuclear reactor facility. Once its operation starts, periodic licensee’s examinations including in-service examination (or maybe called inspection) are done, and then the nuclear regulatory inspection is performed. If the reactor facility is judged not to conform to the NRA’s codes and standards by assessing results of the nuclear regulatory inspection on the periodic licensee’s examination, the NRA may order the licensee to take corrective measures.

Regulations of research and test reactors are provided Article 23 through Item 3–4 of Article 43 of Section 1 of Chapter 4, while those on power reactors are prescribed in Item 3–5 of Article 43 through 3–43 of the same Article of Section 2 of the same chapter. As you can see, Article number of prescriptions on power reactors is irregular, because the regulation structure is drastically changed, reflecting a change in responsibility for regulating power reactors regarding their construction to operation, from the Ministry of Economy, Trade, and Industry to the NRA. Then, the prescriptions on power reactors regarding their construction to operation are added to the Reactor Regulation Act so as to follow those on research and test reactors.

Before 2020, regulations of research and test reactors were slightly different in the framework of examination and inspection at the stages of construction and operation from those of power reactors. By the amendment issued in 2020, reflecting the results of IRRS, the regulations of research and test reactors changed to those of power reactors. Further details will be provided in Sect. 1.5.2.

Table 1.1 Outline of the legal structure for nuclear safety regulations

Stage			Research and test reactors Reactor Regulation Act, Chapter 4, Section 1	Power reactors Reactor Regulation Act, Chapter 4, Section 2
Construction phase	Preparation step	Basic design (including selection of materials)	•Establishment permit (Art. 23) (Permit criteria: Art. 24)	•Establishment permit (Art. 43-3-5) Hereinafter, 43-3 is omitted because of the common number. (Permit criteria: -6)
		Detailed design	•Approval of design and construction plan (Art. 27) •Approval of operational safety program (Art. 37, Sub-art. (1))	•Approval of design and construction plan (-9) •Approval of operational safety program (-24, Sub-art. (1))
	Construction step	Material production	•Pre-service licensee's examination (Art. 28)	•Pre-service licensee's examination (-11)
		Fabrication	•Nuclear regulatory inspection (Art. 61-2-2)	•Nuclear regulatory inspection (Art. 61-2-2)
		Examination, testing and Inspection	•Notification of operational plan (Art. 30)	•Notification of operational plan (Item 17)
	Trial operation	•Decommissioning implementation policy (Art. 43-3-2)	•Decommissioning implementation policy (-33)	
Operation phase			•Maintenance of conformity to technical standards (Art. 28-2) •Periodic licensee's examination (Art. 29) •Nuclear regulatory inspection (Art. 61-2-2)	•Maintenance of conformity to technical standards (-14) •Periodic licensee's examination (-16) •Nuclear regulatory inspection (Art. 61-2-2)
			•Operation plan (Art. 30) •Measures taken for operational safety, etc. (Art. 35) •Compliance with operational safety program (Art. 37, Sub-art. (4)) •Nuclear regulatory inspection (Art. 61-2-2) •On-site inspection (Art. 68)	•Operation plan (-17) •Compliance with operational safety program (-24, Sub-art. (4)) •Nuclear regulatory inspection (Art. 61-2-2) •Periodic safety assessment of continuous improvement (-29) •Approval of operation beyond 40 years of operation (-32) •On-site inspection (Art. 68)
Decommissioning phase			•Decommissioning (Art. 43-3-2)	•Decommissioning (-34)

[Note 1] Section 2 Article 43-3-30, prescribes that a model (or type) certificate should be investigated on design of the type of the specified component relating to nuclear power reactor facilities. And if conforming to the standard of Article 43-3-6, Sub-article (1), Item (iv), the model certificate should be issued and designated in accordance with Article 43-3-31.

[Note 2] Technical assessment on aging is prescribed in Article 14 Sub-article (2) Item (ii) of the "Ordinance on Installation, Operation, etc. of Commercial Nuclear Power Reactors."

[Note 3] In this table, several simplifications are made. In the right-hand side for power reactors, Article 43-3 is omitted because of the common number, and therefore, Article 43-3-6 is simplified to -6.

Figure 1.2 shows how electric utilities, regulatory authorities, and other entities relate to each other at the installation, operation, and other stages of commercial power reactors under nuclear safety regulations after 2020.

Regulations following the license for installation of nuclear facilities are called subsequent regulations.

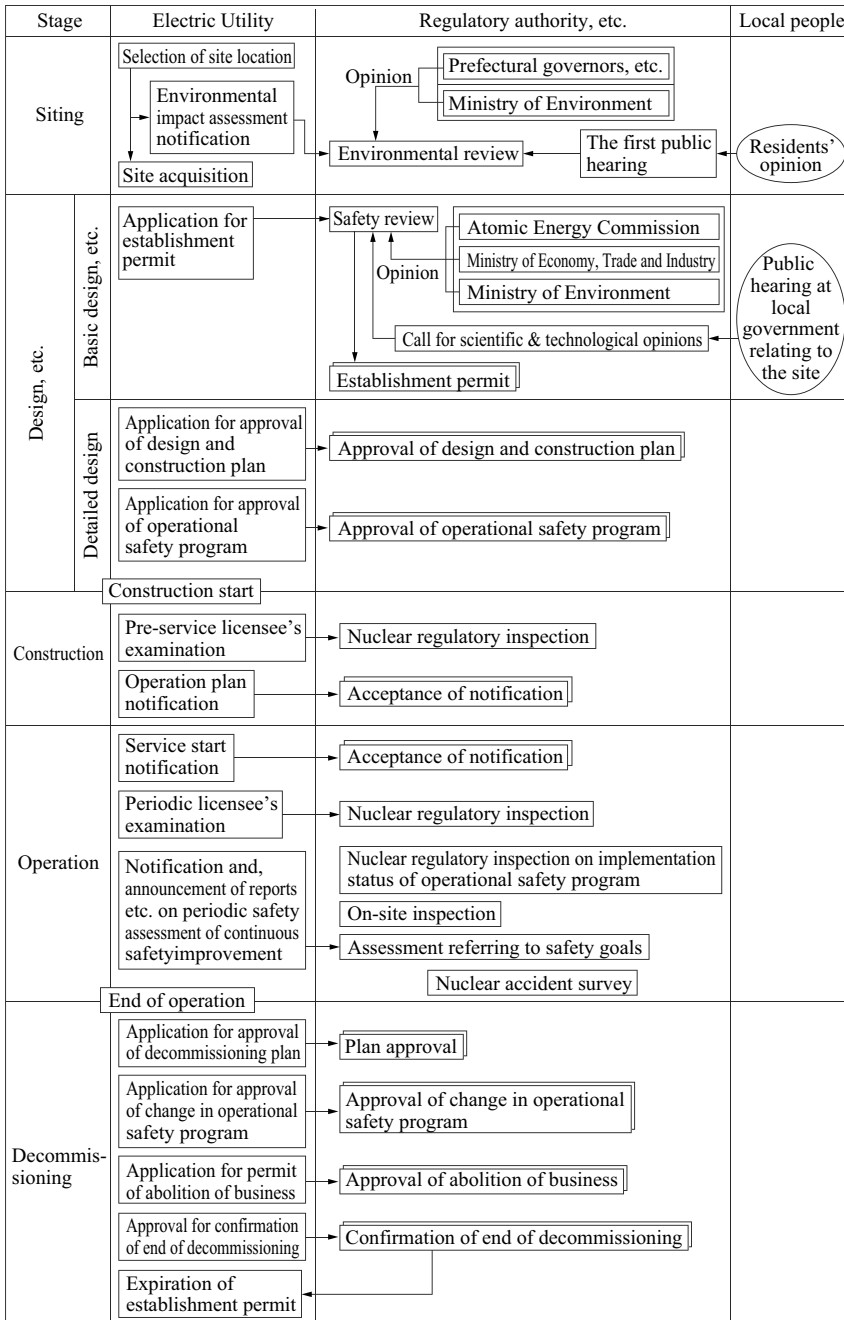


Fig. 1.2 Relations between the owner (business operator or licensee) of the commercial power reactor and regulatory authorities under nuclear safety regulations (additions and corrections are made to Ref. [7])

The installation and operation of nuclear reactor facilities are predicated on that the entity that installs the reactor facility the nuclear reactor installer (owner), or the business operator (in the case of commercial power reactors, or licensee) will verify its safety through an independent safety management system, recognizing the licensee's prime responsibility for nuclear safety. With this as a precondition, the Nuclear Regulation Authority (NRA) enforces all safety regulations, ranging from reactor installation license (or the establishment permit) and operation management to decommissioning, pursuant to the Reactor Regulation Act, for the purpose of securing public safety.

1.5.2 Development of Safety Regulations

[1] Introduction of the safety management review system to electric power generation facilities

(For nuclear facilities, introduction of welding safety management inspection) [8]

The safety level of electric facilities is determined by not only potential hazards dependent on the type and size of the given facilities but also the safety management system of the installer. Recent advances in technology and the penetration of voluntary safety assurance efforts among installers have made the following idea widespread: A more rational regulatory system can be established by promoting safety assurance efforts under installers' own responsibility than through the government's direct examination of electric facilities to verify their conformity to technical standards.

In light of the situation, the "Joint Subcommittee on Electric Power Safety Problems between the Standards and Conformity Committee of the Industrial Structure Council, and the Basic Policy Committee of the Electric Utility Industry Council" proposed a safety management review system under which the government reviews the given safety management system in order to complement the installer's self-examination and encourage the installer to work toward ensuring safety. The system was instituted by the amendment of the Electric Utilities Industry Law in August 1999 and came into force in July 2000.

Here is an overview of the safety management review system.

- (i) Instead of being directly examined by the government for verifying conformity to technical standards, electric facilities for business use shall be subjected to statutory self-examination by the business operator to verify conformity to technical standards, and the operator is obliged to retain records of that examination (obligations to perform statutory self-examination and retain records).
- (ii) The operator is obliged to undergo government review of the self-examination system (including the organization, examination methods, process control) conducted pursuant to (i) (obligation to undergo safety management review).

- (iii) The government shall evaluate the operator’s self-examination system based on the review results and notify the operator of the evaluation results. The operator may reduce the frequency of future review if the notified evaluation results are excellent (awarding of incentive).

The following self-examinations are subject to review:

- (a) Pre-use self-examination (facilities except for those for nuclear power generation).
- (b) Operator’s examination on welding (components related to thermal and nuclear power generation facilities).
- (c) Periodic operator’s examination (only thermal power generation facilities, at first).

Since the establishment of the above rules, at commercial nuclear power reactors, the safety management review system has become applicable to welding inspection. As a result, the operator is required to self-examine its weldments for statutory examination on welding. The government, on its part, must review the operator’s self-examination system. Figure 1.3 shows an overview of these inspections.

The reason for introducing welding safety management inspection to nuclear power generation facilities was the existence of little difference between welding for thermal power components and nuclear components in terms of welding technology, including materials and techniques. With such little difference in technology for welding and weldment examination, the same items and structure for operator’s examination on welding and welding safety management inspection were considered to be applicable. However, since the required quality level of weldments is remarkably

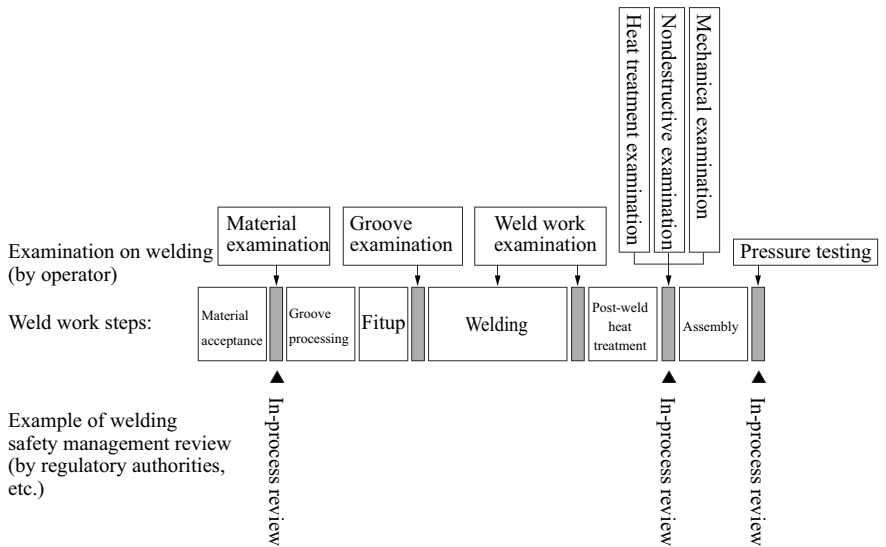


Fig. 1.3 Example of operator’s examination on welding and welding safety management review schedule

different between for thermal power components and for nuclear components, the rules require that the quality level to be applied to each factory for welding, weldment examination acceptance criteria, and the examination method should be tightened according to the importance of the weldment.

[2] Regulatory reform in the aftermath of the JCO accident and scandals

(1) JCO accident [9]

On September 30, 1999, a criticality accident occurred at JCO's uranium processing plant. With respect to the accident, problems were pointed out not only with the company but also with the government.

Company side: Although the direct cause lied in operations using facilities and procedures different from those approved, the following problems were identified as well:

- Incomplete safety management system.
- Lack of safety culture (work efficiency put before safety).
- Lack of awareness of the risk of accidents potentially caused by violations.

Government side:

- Insufficient review of the operation process before approval.
- Insufficient checking of conformance to operational safety programs.

In light of these lessons learned, the government instituted the Nuclear Emergency Act as a special law of the Basic Act on Disaster Management and revised the Reactor Regulation Act to fundamentally reinforce the nuclear emergency preparedness system and the safety regulation structure. In particular, the following revisions were made to the Nuclear Regulation Act:

- Addition of facility periodic inspection to licensee of fabricating and enrichment activity.
- Establishment of a system to inspect conformance to operational safety programs (operational safety inspection).
- Special remarks to require the inclusion of safety education in operational safety programs.
- Appointment of operational safety inspectors.
- Establishment of a system for workers to propose improvements for ensuring safety at nuclear facilities so as to create an environment where they can report to the regulatory authorities more easily when they find violations of safety regulations and other rules.

Information provided under this reporting system led to the discovery of irregularities concerning examination and inspection at Tokyo Electric Power Company (TEPCO)'s nuclear power plants.

Subsequently, at the administrative reform in 2001, the Nuclear Safety Commission and the Japan Atomic Energy Commission went under the umbrella of the Cabinet Office, and the Nuclear and Industrial Safety Agency (NISA) was established.

(2) TEPCO scandals [6, 9]

TEPCO's falsification scandal concerning self-examination records were uncovered by reports (whistle-blowing) in 2000. What followed this was the discovery of another scandal: irregularities with respect to reactor containment vessel leakage rate testing. Factors of these were found in both the company side and the government side.

Company side:

- Dogmatic decisions were routinely made by limited personnel.
- The company made light of the recording and retaining of inspection results and posterior reevaluation.
- There was a lack of consciousness of quality assurance activities.

Government side:

- Self-examination by business operators (or owners, licensee) was left to the decision of each operator, with no regulatory definition provided.
- Methods of assuring the integrity of facilities after the commencement of operation were unclear.

In addition to these, there was a lack of awareness of accountability not only for safety assurance but also for providing scientific and rational grounds.

With these problems in mind, the government enhanced the nuclear safety regulations as follows:

- Establishment of a quality assurance system and maintenance management activities.

Regarding the quality assurance system, a legal step was taken to oblige operators to stipulate quality assurance activities in their operational safety programs, while requiring the government to check their implementation status through operational safety inspection. Quality assurance activities shall be based on international standard ISO 9001:2000.

- Introduction of the aforementioned system for periodic operator's (or licensee's) examination.
- Statutory obligation to conduct periodic safety review.
- Introduction of integrity assessment.

Operators have become obliged to conduct integrity assessment during periodic operator's examination and report the results to the government. Rules were established on how to evaluate the integrity of equipment if any crack should be found.

- Identification of facilities whose construction plans must be approved.

Furthermore, the Japan Nuclear Energy Safety Organization (JNES) was set up.

[3] Improvements of inspection systems [10]

Since the launch of the above inspection system in 2003, the NISA has been promoting its steady use in the industry and been searching for the ideal form of the inspection system that can address the aging of nuclear power plants, by resuming the “Study Group on Inspection.” As a result, NISA proposed the direction of inspection system improvement that focuses on the following three points:

- Introduction of an inspection system for safety activities pursuant to the “safety program.”
- Introduction of a safety system centering on actions important to safety assurance.
- Development of guidelines for route cause analysis.

For the “introduction of a safety system centering on actions important to safety assurance,” operational safety inspection after shutdowns for periodic inspection and inspection in the event of deviation from operational limits was introduced in September 2007. For “development of guidelines for route cause analysis,” requirement for root cause analysis came into force in December 2007. “Introduction of an inspection system for safety activities pursuant to the ‘safety program’” was fulfilled in 2008, giving flexibility to inspection and examination followed by reactor shutdown and making it possible to flexibly schedule reactor operation intervals and government periodic inspections based on the reactor’s operation performance achievements (maximum permissible intervals of 18 and 24 months, in addition to the conventional setting of 13 months).

[4] Relations between periodic inspection, periodic safety management review (inspection), and operational safety inspection

Periodic inspection, periodic safety management review, and operational safety inspection are all conducted by the regulatory or administrative agency during the operation phase. They have different scopes of application and inspection items that are not overlapping.

Periodic inspection: Items of major importance among those included in periodic operator’s examination conducted by the business operator are inspected. This includes review of the process of inspection.

Periodic safety management review: The implementation structure and other aspects of periodic operator’s examination are reviewed.

Operational safety inspection: The operator’s overall maintenance and examination activities are examined from the viewpoint of quality assurance.

[5] Drastic regulatory reform in response to lessons learned from the TEPCO Fukushima Daiichi nuclear accident

In addition to facility damages due to the 2011 Tohoku Oki Earthquake and huge tsunami generated by this earthquake, the severe accident at the TEPCO Fukushima Daiichi nuclear power station that occurred on March 11, 2011 caused exposure to citizens living in the nearby areas by discharged radioactive materials (fission

products) as well as an environmental damage in the surrounding areas and created tens of thousands of displaced citizens.

Many reviews relating to this accident have been published. Here, an outline of this accident is introduced, referring to Ref. [11]. Next, the regulatory reform is mentioned.

(1) Outline of the earthquake, tsunami, and accident

The 2011 Tohoku Oki Earthquake, measuring the moment magnitude (M_w) of 9.0, struck Japan. The maximum acceleration amplitude of the earthquake at the site of the Fukushima Daiichi nuclear power station was at the level of design basis earthquake ground motion S_S for seismic design. The station consists of six BWRs. The units 1, 2, and 3, which were operating at full power at that time, were normally shut down due to the earthquake. Although some distribution towers and other facilities were damaged by the earthquake, resulting in a loss of all off-site electric power sources to the site, emergency diesel generators normally started to provide AC power to emergency systems, and the reactors were cooled down as planned. However, their turbine buildings and other facilities were flooded by a large-scaled tsunami generated by the earthquake. Consequently, an extended station blackout (SBO, loss of all the station's AC sources) was caused, because many safety-related electric components such as water-cooled emergency generators, switch boards, DC batteries were located on the first floors of the buildings.

Eventually, the units 1–4 completely lost the supply from the electric sources, excluding the units 5 and 6, and then failed to transfer nuclear-generated heat (actually, residual heat) to seawater, namely the final heat sink. The failure is called a Loss of Ultimate Heat Sink (LUHS). As a result, core meltdowns occurred at the units 1 to 3, and massive volumes of hydrogen were generated mainly by the chemical reaction of Zirconium alloy with water (or steam). Hydrogen explosions occurred in the reactor buildings of the units 1 and 3, finally causing ground-level releases of radioactive materials. The hydrogen generated in the unit 3 was inferred to flow into the unit 4 through connecting piping systems between both units, subsequently producing an explosion in the unit 4 reactor building. In the unit 2, an explosion did not seem to occur, because a part of blow-out panels happened to open, but radioactive materials were discharged into the environment. At the unit 6, one air-cooled emergency diesel generator was not flooded and then succeeded in supplying the electric power, and since cross-connections were available between the units 5 and 6, later supplied the power to the unit 5. In the end, both units succeeded to shut down to the safe state.

(2) Regulatory reform reflecting lessons learned from the accident

This severe accident reminded us of the following once again:

- There is likelihood that a severe accident beyond design-basis accidents that were selected as the basis for safety design.
- Such a severe accident may occur due to multiple failures of safety functions that are maintained by means of safety-related facilities for preventing the occurrence

of the design basis accidents, preventing evolution toward severe accidents, and mitigating (or limiting) their consequences. Here, an accident to cause a severe damage of reactor core as beyond the design basis accidents is called a “severe accident.”

The following safety measures were recognized to be necessarily taken as safety regulations in Japan [12].

- (i) To prevent a severe accident by protecting the reactor core, etc. from a severe damage.
- (ii) Even in an unlikely event to cause a severe accident, to prevent any failure of a reactor containment vessel and an abnormal release of radioactive materials outside the reactor facility site.
- (iii) Assuming an intentional failure of the containment vessel, to suppress dispersion of radioactive materials.
- (iv) To cope with a large-scale collapse of a nuclear facility due to a huge natural hazard or a terrorism such as intentional crash of a large aircraft.

In order to reflect lessons learned from the Fukushima Daiichi nuclear accident and its cause investigation results from all aspects, not limited to necessity of measures against the severe accident from the above-mentioned aspects, reviews and investigations of the accident were done in Japan and abroad, including those on fundamental concepts and frameworks of safety regulations. These achievements were summarized as final reports of various committees; for example, in Japan, the final report of “Investigation Committee on the Accident at the Fukushima Nuclear Power Stations of Tokyo Electric Power Company,” issued in 2012. The committee was set up by the Japanese Government. Recommendations on Policy, etc., which were pointed out in their final reports in Japan and overseas, led to safety regulation reform and related changes. Based on results of the reviews and investigations summarized by those committees, the Nuclear Safety Commission (NSC) and the NISA, which were the safety regulation organizations at the time of the Fukushima Daiichi nuclear accident, conducted investigations on safety regulations, aiming at reflecting the lessons learned from the accident to the safety regulation reform. The following are key investigation results [13].

- (1) Establishment of the Nuclear Regulation Authority having a high level of independence and transparency

A commission or authority responsible for nuclear regulations was judged to need to ensure a high level of independence and transparency, separating from governmental bodies for nuclear utilization and promotion. In September 2011, the Nuclear Regulation Authority was established as an independent organ of the Environment Ministry, a so-called “Article 3 Committee” under Article 3 Subarticle 2 of the National Government Organization Act, that is a committee having independence from politics, economic policy and other governmental organizations, and regarding authorities and human resources, as well as independent from business operators as a matter of course.

Before the Fukushima Daiichi nuclear accident, the NISA under the Ministry of Economy, Trade, and Industry (METI) was responsible for safety regulations of nuclear power reactors. The NSC implemented double-check (secondary review) on safety review, but the NISA only was responsible for the safety regulations on construction work plan and the subsequent stages. Under the Reactor Regulation Act that was drastically changed after the Fukushima Daiichi nuclear accident, the NRA conducts the safety regulations on the construction work plan and the subsequent stages in an integrated way as well as safety review, as separated from the METI. Concerning the safety review, the double-check system (namely, the secondary review after the NISA's review) was abolished, and accordingly, the NRA solely conducts the safety review.

With this change, ordinances and guides necessary to perform safety regulations have been developed by the NRA [14]. Those that were developed by the NSC are referred to case by case [15].

(2) Strengthening of countermeasures against severe accident

Technical aspects of the safety regulatory reform are mentioned above. Regarding countermeasures against a beyond-design-basis accident, licensees are recommended to voluntarily take accident management (AM) measures. The detail of such AM measures was cited in the guide entitled "On Accident Management as severe accident countermeasures to be taken for power-generating light-water type nuclear facilities" that was issued by the NSC, in May 1992. From experience of the Fukushima Daiichi nuclear accident, the AM countermeasures were changed to be legally required, and the related design requirements were strengthened [16]. It is better to note here that in the above-mentioned guide, a severe accident was defined as an event at which plant behavior parameters such as temperature or pressure drastically exceed those at design basis accidents and adequate core cooling or reactivity control by means of measures taken as safety design fails, leading to a severe damage of reactor core.

To take measures for the severe accident is required as the fourth-level safety measures in the defense-in-depth concept according to the recommendations by the IAEA. The main objective of the severe accident measures is to avoid or minimize contaminations outside the site of nuclear reactor due to the accident. It is required to practically exclude off-site contaminations by significantly reducing likelihood of an accident sequence to cause an early or large radioactive release [17]. The NRA has set a safety goal as the target to exclude the likelihood of contamination. Note that this safety goal is not a standard but that is defined as a target aiming at realizing safety regulations [18]. From a viewpoint of excluding the contaminations of environment, a total release amount of Cs-137 should be limited below 100 TBq in order to avoid a long-term evacuation of citizens in the surrounding of the nuclear power station (An additional requirement that an occurrence frequency of a severe accident at which the release amount of Cs-137 would exceed 100 TBq should be below once in 10^6 reactor-year of operation) [19].

(3) Strengthening management plans for nuclear disaster

In the Japanese regulations, an emergency plan, etc., which prescribes responses to an emergency situation on- and off-site of a nuclear power plant and is required as the fifth-level safety measures in the defense-in-depth concept, should be developed in accordance with the Basic Act on Disaster Management and the Nuclear Emergency Act but not the Reactor Regulation Act, because the fifth level is related to the disaster [20]. Article 1 of the Nuclear Emergency Act, which plays a central role, prescribes the following:

The purpose of this Act is to strengthen nuclear disaster countermeasures, in cooperation with the Reactor Regulation Act, the Basic Act on Disaster Management and other Acts regarding nuclear disaster prevention, by providing special measures for the obligations, etc. of nuclear business operators (licensees) relating to nuclear disaster prevention, the issuance of a declaration of nuclear emergency situation and the establishment, etc. of nuclear disaster response headquarters, and the implementation of emergency response measures and other items relating to a nuclear disaster, taking into account of the particularity of a nuclear disaster, thereby protecting the lives, bodies and properties of citizens from a nuclear disaster.

At the national government level, the National Disaster Management Council must formulate a “basic disaster management plan,” and correspondingly, at the level of a prefecture as a local government, a “prefectural area disaster management plan” must be formulated, and also at the level of a municipality, a “municipal area disaster management plan” must be formulated. The NRA also is required to formulate a nuclear disaster countermeasures guideline on professional and technical matters so as to ensure smooth implementation of nuclear disaster management and others by business operators (or licensees), the national government, local governments, and so on. This guideline amended the former Nuclear Emergency Act, which was formulated before the Fukushima Daiichi nuclear accident, so as to drastically enhance and strengthen it by incorporating new concepts on evacuation and others.

The above-mentioned regulatory systems on nuclear disaster management were significantly reinforced, reflecting lessons learned from the Fukushima Daiichi nuclear accident, although essential parts of the systems were established before the accident.

(4) Introduction of operational period limitation to 40 years [21]

In the process of safety regulatory reform, the operational period of nuclear power plants was determined to limit to 40 years as calculated from the day of passing a pre-service inspection at the first time. If the business operator (or licensee) submits the application for operation beyond 40 years and then the NRA approves the extension, it is allowed to extend the operational period within 20 years only for one time.

(5) Strengthening measures against natural hazards such as earthquake and tsunami (or external event measures) [22]

Before the Fukushima Daiichi nuclear accident, in the review guide, entitled the “Review Guide of Safety Design for Nuclear Power Generating Light-water type Reactor Facilities,” (partially amended in March 2001) Article 2 “design considerations on natural phenomena” stipulated the following:

1. “Structure, systems and components having safety functions should be classified in the seismic design, by taking into account importance of the safety function and safety impacts at the event of loss of the functions due to an earthquake, and should be designed to withstand design seismic forces to be appropriately determined.
2. Structure, systems and components having safety functions should be designed to withstand without damaging safety against assumed natural phenomena other than earthquake. Structure, systems and components having an especially high level of safety functions should be designed withstand appropriate combinations of accident loading and the most severe conditions of assumed natural phenomena or naturally induced forces.”

Concerning a tsunami, slope collapse, etc., in the surrounding of a nuclear reactor facility as an accompanying phenomenon of an earthquake (i.e., seismically generated tsunami, slope collapse, etc.), proper measures are required to be taken, as prescribed in Article 8 “considerations on accompanying phenomena of an earthquake” of the “Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities” (determined by the NSC, September 2006) (hereinafter referred to as the “Seismic Design Review Guide”). It provided the following rules on tsunami.

“(2) Safety functions of a facility should not be severely damaged due to a tsunami whose occurrence is extremely rare but appropriate to assume because there is likelihood of its occurrence during a service life of the facility.” In the “Instructions of safety review regarding seismic safety of nuclear power reactor facilities” (approved by the NSC, in December 2010) that was formulated based on the Guide, tsunami-related issues were addressed as follows: “2. Safety assessment against tsunami”.

It should be confirmed that water level fluctuation, sand movement, etc. to be caused by the tsunami whose occurrence is extremely rare but appropriate to assume, because there is likelihood of its occurrence during the service life of the facility (hereinafter, the tsunami is referred to as assumed (or evaluation) tsunami.) do not have any serious effects on safety functions of the facility.”

Based on experiences of the 2011 Tohoku Oki Earthquake, huge tsunami followed the earthquake, and the Fukushima Daiichi nuclear accident, seismic design-related parts of the aforementioned Guide and Instructions were reviewed for changes. The NRA also actively devoted itself to such investigations, particularly because the tsunami was the main cause of core meltdowns of the units 1 to 3 at the Fukushima Daiichi nuclear accident. In the investigations, discussions were made from aspects on what measures should be taken to minimize the tsunami risk, because there is likelihood of occurrence of such a tsunami that may exceed the evaluation tsunami which was established on the basis of the latest scientific knowledge [23]. As a result, hierarchical tsunami measures are required to take. A “dry-site” concept to prevent the site from run-up or flooding due to the design basis tsunami should be the basis of the measures, and further, a certain level of resistance should be maintained even for an event beyond the design basis (for example, an event at which a tsunami may cross the seawall and then flow into the site.) [24]. An explanation on how the seismic

design and tsunami-resistant design have been changed in the response to lessons learned from the Fukushima Daiichi nuclear accident is given in Sect. 7.4, Chap. 7.

[6] Reform of safety regulations, particularly regulations for inspection, in response to the IRRS by the IAEA.

The regulatory systems with regard to inspection for nuclear facilities, especially for nuclear power reactor facilities, have been continuously advanced aiming at highly effective regulations, as mentioned in [1–3] above. As a peer review of the safety regulation systems in the Japanese Government, an Integrated Regulatory Review Service (IRRS) was done by a team organized by the International Atomic Energy Agency (IAEA), in June 2007 and in January 2016. At the IRRS conducted in 2016, a wide range of safety regulation issues, including the regulation system for inspection, were reviewed, and many advices were made. In response to these advices and recommendations, the NRA was required to develop a new mechanism of safety regulations from viewpoints of realizing more effective safety regulations, and therefore, amended the Reactor Regulation Act, which was enforced in 2020.

In this textbook, describe key points of inspection-related issues, referring to Ref. [25]. A detail of the amended inspection regulation is discussed in Chap. 9.

Before 2020, regulatory requirements on inspection including the Reactor Regulation Act and Electricity Business Act were based on a staged (or phased) approach. Unless at each stage of construction work, safety is sufficiently confirmed, and it is regulated to prohibit proceeding to the subsequent stage. And as described below, regulatory organizations are directly concerned at the stages of confirming compatibility of facilities with technical standards and so on. Further, they checked implementation status of business operator's (or licensee's) operational safety activities (see [Before 2020] on the left side of Fig. 1.4).

- Inspections by regulatory organizations in order to confirm the conformity of facilities to standards, and so on:
 - Pre-service inspection, fuel assembly inspection, welding inspection for research and test reactor facilities, etc., and periodic facility inspection
- Inspections and reviews by regulatory organizations in order to check the implementation status of business operator's operational safety activities:

Welding safety management inspection (review), periodic safety management review, and operational safety inspection (welding safety management inspection (review) and periodic safety management review are done for mainly checking the implementation frameworks of welding operator's examination and periodic operator's examination that are performed by operator (or licensee), respectively).

Note that concerning welding examination/inspection of nuclear power reactor facilities, manufacturers like welding operators perform the primary examination. Business operators (or licensees) of nuclear reactors such as a power generation company's operator check conformity to standards as the secondary examination. As the final inspection (or review), regulatory organizations perform welding safety

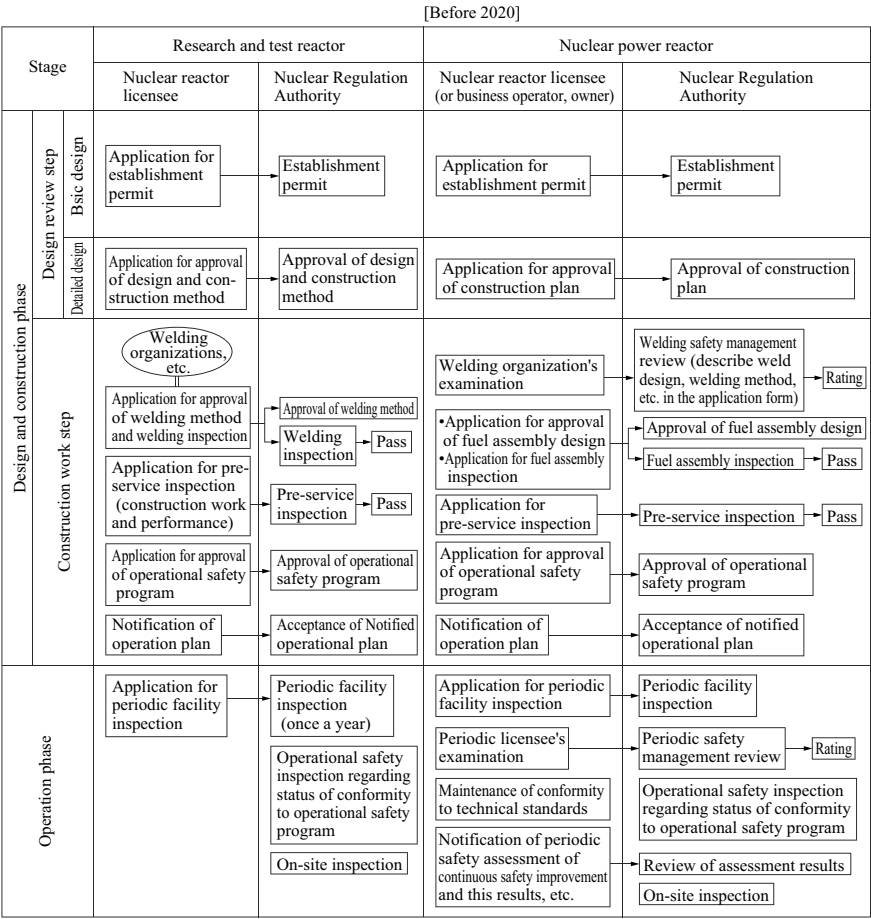


Fig. 1.4 Revolution of inspection regulation based on IRRS by IAEA

management review. On the other hand, concerning research and test reactor facilities, any amendment along with the regulatory reform as mentioned in [1] above was not done, and therefore, concerning welding examination/inspection, a regulatory organization directly performed welding inspection after welding operators conducted the primary examination.

In the IRRS, advices and recommendations were made on such situations. Regarding examination/inspection, the following and others were advised and recommended [26]:

- Welding inspections for research and test reactor facilities, etc., other than nuclear power reactor facilities and fuel assembly inspections were carried out by the NRA, and the scope and content of their inspections were fixed by the NRA (actually, by technical specialists) in advance. This situation provides confidence

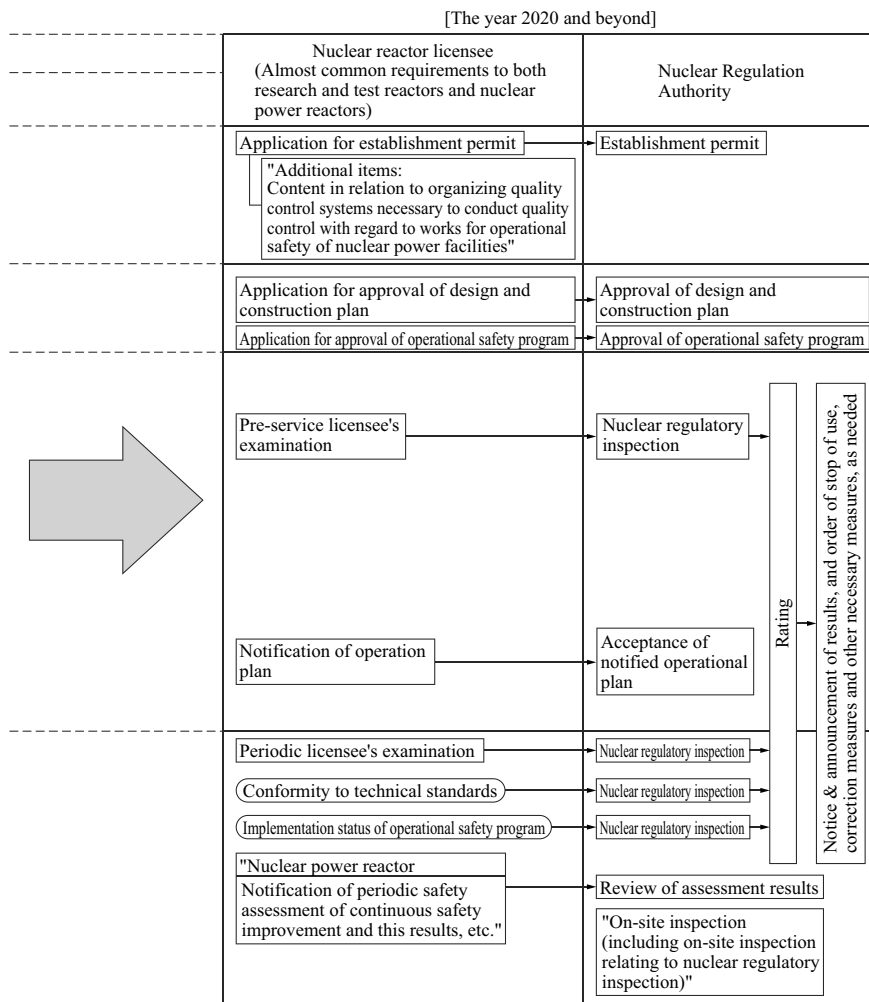


Fig. 1.4 (continued)

in the integrity of system welds important to safety. In this regulation, the responsibility of assuring the integrity of welds lies with the regulatory organizations, but not with the business operators or licensees.

- Before the IRRS's recommendations, the periodic facility inspection was done only by the NRA or by the equivalent organization. However, the NRA could not start the inspections before the business operator applies for the NRA's inspections to the NRA. The IRRS pointed out that it was unusual.
- The scope and content of operational safety inspection were strictly prescribed.
- Such strongly specialized and prescriptive examination/inspection might diminish prime responsibility of the business operators (or licensees) for safety.

- Based on such advices and recommendations by the team of the IRRS, the IAEA made the following recommendations [26].

“The government should improve and simplify the inspection framework to:

- Increase the NRA flexibility to provide for efficient, performance-based, less prescriptive and risk-informed regulation of nuclear and radiation safety,
- Allow the NRA decisions about reactive inspections to be made at the lowest possible level.

Based on the revised inspection framework, the NRA should develop and implement a program of inspection of all facilities and activities specifying types and frequency of regulatory inspections (including scheduled inspections and unannounced inspections) in accordance with a graded approach.”

In addition, the following was suggested [26]:

The NRA should consider in providing training and retraining of its inspectors in order to improve their competencies for inspections, associated assessments and decision making.

In accordance with such advices, recommendations, and suggestions, the NRA modified the inspection framework based on the following philosophies and concepts, aiming at realizing more effective regulations:

Basic philosophies;

- To attain operator’s (or licensee’s) prime responsibility for ensuring safety, and to continuously improve safety according to own independence.
- To realize a higher level of safety by both efforts of operators and regulatory organizations.

Concepts of regulations for inspection;

- The regulatory structure to perform the following duties as the regulatory organization needs to be developed.
- It should be thorough that the prime responsibility for safety lies on operators, and that the regulatory organizations may not substitute the said responsibilities in the extent.
- Improvements should be requested as administrative measures if insufficient matters are confirmed throughout the operational safety activities as well as in checking standards, etc., established to meet regulatory requirements, and conformity to the corresponding standards at turning points in a staged approach.
- Subject to the proper implementation of operator’s initiatives to ensure safety of a nuclear facility, the regulatory authority is required to urge the operator to find and tackle the issues of his/her own accord with the questioning attitude everytime and without limiting the scope, content, and so on.
- Clarifying that the operator has the responsibility for confirming the conformity to the content that has been approved by the regulatory organization and regulatory standards, etc., the regulatory organization requires to the operator to assess and improve of his/her own accord the adequacy of whole operational safety activities

including the confirmation activities concerned. In addition, it is thought to be appropriate for the regulatory body to comprehensively monitor the operator's whole operational safety activities, to grasp their real situations and to assess that the operator accurately identify the matters to be improved and link the improvement activities to the safety enhancement.

- If as a result of monitoring and assessments, insufficient matters are found on the operational safety activities, for example, insufficient confirmation of conformity to regulatory standards, it is required to urge the operator to improve the operator's operational safety activities and then realize a higher level of safety assurance, not only correcting nonconformity and violation, by changing to such a structure that assessing impacts on safety and correction effects, administrative measures are flexibly taken if necessary.
- It is important to have objective indices relating to a level of safety assurance and to confirm the situation of taking effective measures for enhancing the level. Measurable indices need to be identified, in taking a risk-informed and performance-based approach. The performance-based approach means to reflect achievements on safety assurance resulted from operator's operational safety activities.

According to the aforementioned philosophies and concepts, the Reactor Regulation Act was amended to those based on the following framework:

(1) Thoroughness of operator's prime responsibility for safety

- Actions for confirming the conformity to standards that were legally documented for inspections and confirmation to be conducted by regulatory bodies should be obligated to be voluntarily taken by the operator in order to persist in operator's prime responsibility for safety assurance.

At a construction stage, the operator should implement pre-service examinations, fuel assembly examinations and welding examinations as pre-service operator's examinations, while the regulatory body should conduct nuclear regulatory inspections by which implementation frameworks, etc. of the operator are reviewed.

- All the inspection regulations that before the IRRS, the regulatory organizations had done for confirming the conformity to the standards and approved content by periodic facility inspections, etc., are carried out in the course of auditing and assessing the implementation status, results, framework, process, etc., of examinations and confirmation that are done by the operator.
- Unification of entities relating to obtaining the approval and implementing the inspections

It should be noted here that examinations done by a licensee (or an operator as mentioned above) at the stage of manufacturing (or fabrication) before the commencement of operation or service, and inspections carried out by a regulatory organization as the third party at the same stage, including a welding examination/inspection and preoperational inspection, are not thought to be definitely termed.

Japanese regulatory organizations and licensees use the term of “pre-service examination/inspection” as the above examinations/inspections, in place of preoperational examination/inspection or construction examination/inspection. Here, it should be noted that the “pre-service inspection (PSI)” is generally termed as the inspection implemented for identifying the initial conditions for the in-service inspection (ISI). In this textbook, examinations and inspections carried out at the stage of construction, excluding nuclear regulatory inspections, are referred to as “pre-service examinations and inspections.”

(2) Incorporation of new frameworks for audit and assessment

- For whole operational safety activities, which are operator’s duties, the regulatory organizations should comprehensively audit and assess, without limiting the time and duration, the implementation status and operator’s initiatives in continuous improvements from the permit of business through the completion of decommissioning.
- In order to clarify targets for the above-mentioned audits and assessments, activities concerned with an operational safety program include those at the stages of design and construction.

Approval of operational safety programs were previously allowed to obtain before the commencement of operation, but are amended to obtain before the start of construction works for installation.

- Quality assurance activities, which are treated as a part of operator’s (or licensee’s) ability, should be made from the stage after receiving an establishment permit, and should be included as targets for audits and assessments. Therefore, these activities are registered in the establishment permit application.

(3) To hold confirmation function by regulatory organizations

- Even if conformity to standards and licensed content is implemented as operator’s duties, frameworks for step-by-step confirmation by regulatory organizations should be maintained, on the basis of a staged approach (completion of confirmation made by the regulatory organizations at the end of each stage forms a pause in the staged regulation).
- Methods for confirmation by regulatory organizations should include witness to the operator’s examination, confirmation of examination records, etc., but may not include inspection by the regulatory organizations that repeats the examination by the operator or that is thought to substitute for the operator’s responsibility.

In accordance with the above-mentioned new mechanisms of safety regulations, the Reactor Regulation Act and the related Acts have been revised as shown in the right-hand side of Fig. 1.4 and then enforced from 2020.

1.6 Recent Waves in the Concept of Structural Standards

As explained in Sect. 1.4, structural standards have evolved while embracing advances in technologies needed for fabricating engineering equipment. This can be described as technology-driven development. Technology innovation will continue ceaselessly in the years to come and must continue. Structural standards should advance along with this innovation. However, in future, this mechanism will be reversed occasionally. Innovation of standards triggered by structural standards themselves will take place, calling for technology innovation. For example, this may happen like this: Innovation occurs to the concept or notion of assuring structural integrity (e.g., in a system based code, to be mentioned later) and raises the need for the development of new technologies, such as those for examination. A kind of outside (or foreign) pressure to reform standards is a reform of safety regulations as mentioned in the previous section. There are already signs of conceptual change in structural standards. With this situation in view, this section briefly explains what kinds of technology developments have been made, and which direction structural standards are going at large. It is recommended that when studying individual technologies in Chap. 2 onward, you examine how those technologies are likely to change amid such a general trend in structural standards.

1.6.1 Adoption as International Standards and Regulation Standards Moving Toward Performance-Based Criteria

In recent years, standards have been in the throes of turbulent change as a consequence of the profound globalization of the economy. According to a report by the Japanese Industrial Standards Committee [27], developed countries in the West are trying to secure their competitive advantage in the international market by strategically using standards and making their own standards adopted as international standards, in an effort to disseminate technologies developed in their countries (or regions). Meanwhile, diversifying consumer needs are increasing the importance of standards as objective decision criteria.

In Japan, unlike these countries or regions, authorities related to safety regulations have been concentrated on regulatory agencies, which have defined detailed technical requirements as technical standards (these standards set by regulatory authorities are called enforcement rules or regulation standards). This has resulted in delays in actions to proactively and flexibly incorporate the latest technologies, as well as in excessive regulations arising from the tightening of regulations following every major accident. However, more recently, Japan has come to strongly recognize the need to aim at establishing Japan's original technologies as international standards and thereby increasing Japan's industrial competitiveness. Japan has become aware that this requires allowing technical standards to be revised promptly in accordance

with technological advances and constructing systems consistent with their international equivalents. The idea behind this is so-called the standards war, where “those who dominates standards will dominate the market.” In this war, standards that can survive as international standards are those that help meet technical targets (e.g., component quality levels) conducive to a balance between safety and economy, while allowing safer and less expensive production of goods (or more simply, production of better quality goods) through the combination of Japan’s cutting-edge technologies and knowledge. International competitiveness can be obtained only by successfully combining state-of-the-art technologies and knowledge with standards.

In 1994, the World Trade Organization (WTO) issued the Agreements on Technical Barrier for Trade (TBT Agreements), which require countries to make their standards consistent with international standards. Since then international consistency has become a requirement for local standards. This has led to the recognition that standards should be in hierarchical structure and that upper standards should reflect the latest knowledge and set technical targets (so-called performance-based criteria), in addition to being consistent to international standards. Lower standards, or detailed standards, should stipulate the details of upper standards. They should allow diversity in the means to fulfill technical targets, while reflecting cutting-edge technologies and knowledge and permitting their use.

Under these concepts, Japan is moving toward the situation where government-set regulation standards merely stipulate proper performance-based criteria, instead of covering prescriptive criteria, while the development of prescriptive criteria needed for actual production and utilization is delegated to private standardization bodies. This allows the latest technologies and knowledge to be quickly and more easily incorporated in detailed standards.

Next, how performance-based criteria are developed about structural standards under safety regulations in Japan is explained in detail. Before discussing performance-based criteria, please remember the history of improvements of the nuclear safety regulations in Japan. In the period of the NSC, which was reorganized to the NRA in response to the Fukushima Daiichi nuclear accident, the safety regulations may be summarized as shown in the left-hand side of Fig. 1.5. For commercial power reactors, establishment license based on safety review is regulated by the Reactor Regulation Act, while details of regulation for processes from detailed design to periodic inspection are given in the Electricity Business Act. The Electricity Business Act set forth technical targets to be met by commercial power reactors as technical standards and required that construction plans, pre-service inspection, welding methods, examination, and maintenance should conform to, or not fail to conform to, these standards. As Fig. 1.5 shows, these technical standards are established as ministerial ordinances.

Before establishing a performance-based criteria system, the technical targets described in these ordinances of the Electricity Business Act had been somewhat vague then. Nevertheless, the system of technical standards were effectively functioned, because the regulatory standards included both of a group of ordinances that stipulated technical performance targets and a group of notifications, positioned as prescriptive criteria, that stipulated detailed technical requirements, and therefore,

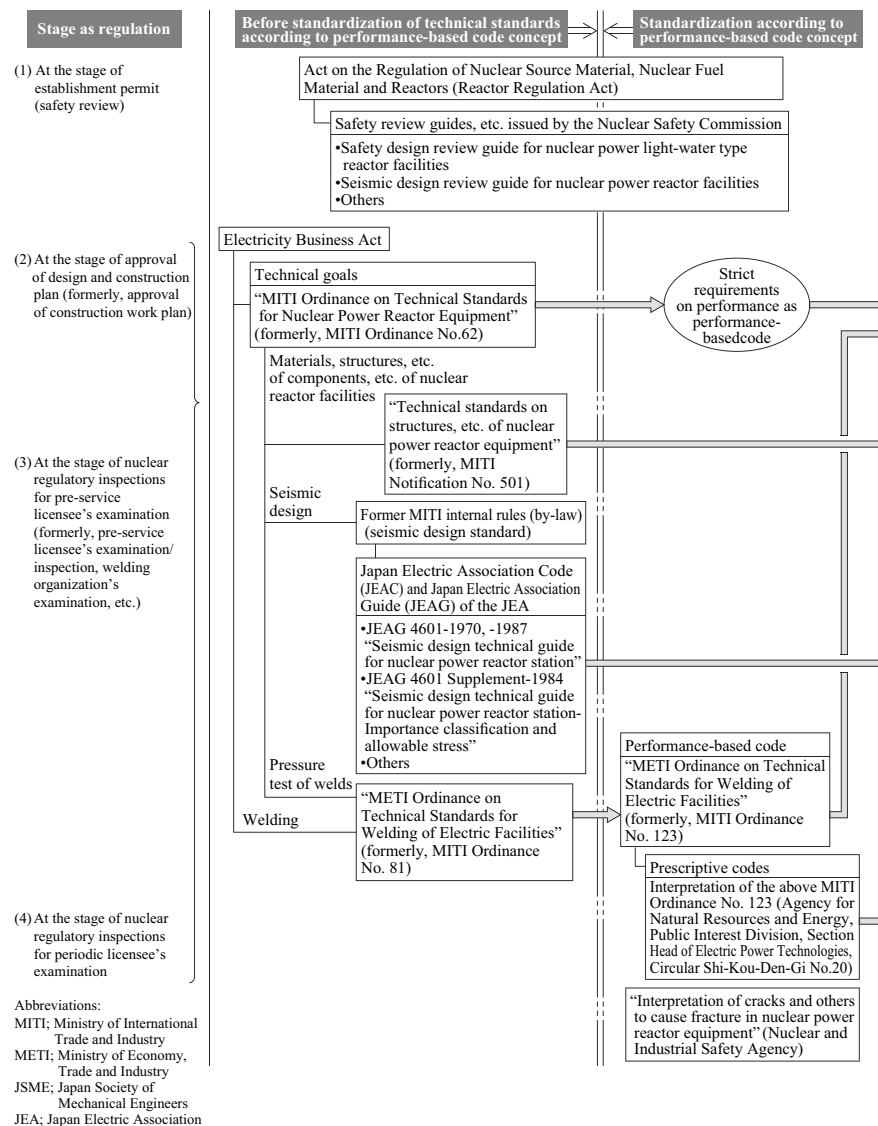


Fig. 1.5 Historical development of technical standards: framework of performance-based codes and prescriptive codes regarding structural integrity of nuclear power reactor facilities

the vagueness of the ordinances was supplemented by the regulated notifications. The ordinances included the Electricity Business Act. After the establishment of performance-based criteria, the ordinances were revised so as to accurately stipulate technical targets, because it was no longer possible to rely on ministerial notifications to define detailed technical requirements, or prescriptive criteria.

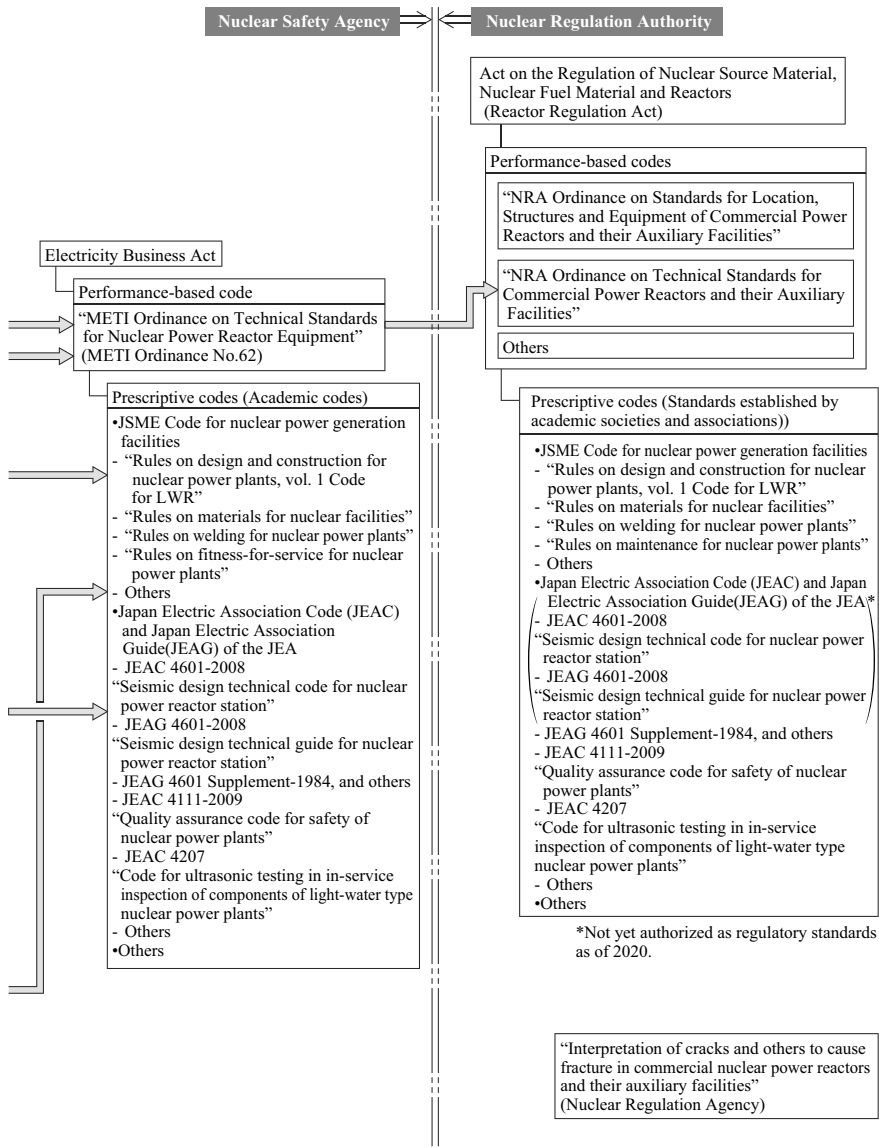


Fig. 1.5 (continued)

Consequently, these prescriptive criteria, or standards specifying detailed technical requirements were established by academic societies, etc. and they have been cited as statutory requirements.

Afterward, the NRA was established and then has taken the full responsibility for the safety regulations, correspondingly leading to having the overall responsibility for technical standards. The NRA has taken over the hierarchical code structure, namely, the ordinances, as the regulatory standards and also the upper standards, that are functioned as the strict performance-based criteria, and the codes and standards that are established by academic societies and associations, as the lower standards and the prescriptive criteria.

Note that the Electricity Business Act has been still functioned as technical standards for assuring structural integrity of electric equipment, because the METI, the successor of the MITI, continues to take the responsibility for electric facilities, except for the nuclear safety.

1.6.2 Performance-Oriented Quality Assurance Standards

To guarantee that the quality of facilities and components actually meet the technical requirements and standards, it is essential to not only inspect and review the resulting (or produced) components but also examine the process of their construction. Personnel (engineers) in charge should act appropriately in each step of the process, and the organization should be designed to facilitate the execution of this. Quality assurance standards stipulate rules that focus on human activities and organizational structure that have an impact on quality like this. Basically, quality assurance standards are requirements for developing a quality assurance plan and implementing it.

The first guidelines for the quality assurance of nuclear power plants are “Appendix B Quality Assurance Criteria for Nuclear Power Plant” [28] of the Code of Federal Regulations (10 CFR 50) in the U.S.

Based on this code, the IAEA established quality assurance standards in 1978. In Japan, the Japan Electric Association (JEA) issued standards pursuant to these in 1981 as JEAG 4101.

Recognizing recently that quality assurance activities had fallen into formalism, such as putting too much emphasis on documentation, the IAEA revised its quality assurance standards (50-C/SG-Q) in 1996 in an attempt to make them provide for effective quality assurance activities (performance-based quality assurance) [29]. In Japan, JEA revised JEAG 4101 in 2000 for the same purpose [30]. Subsequently, quality assurance activities became subject to operational safety inspection, which required the clarification of quality assurance requirements for nuclear safety. Following a proposal by the Study Group on Inspection, under the Nuclear and Industrial Safety Subcommittee, that quality assurance requirements should be developed based on ISO 9001:2000, JEA issued “Quality Assurance Code for the Safety of Nuclear Power Plants (JEAC 4111–2003)” [31], which was built on ISO 9001 and aligned with the concept of the IAEA’s 50-C/SG-Q.

Here is an example of problems whose identification leads to the realization of the need for performance-based quality assurance and ways to make quality assurance more effective. When it is found that a process is three days overdue, the organization may decide to suspend quality assurance activities because of the failure to meet the due date even if it poses no quality assurance-related problems. However, such decision should not be entirely based on the checklist results but rather made flexibly by the responsible personnel. A decision to permit the process as long as there is no significant problem would be more desirable (although the decision on permission may have to be justified). Effective quality assurance activities are possible by clearly defining the responsibility of the personnel in charge and respecting that personnel's decision.

Quality assurance standards are related to human factors, or institutional aspects, as mentioned above. Other standards concerning human factors include certification and accreditation standards for certifying the competence of engineers involved, as listed in Fig. 1.1. What is called certification in the ISO standards refers to third-party attestation (proof) concerning personnel, products, systems, and so on. Accreditation is third-party attestation that organizations, especially conformity assessment bodies, have officially demonstrated their capability. In Japan, these human-factor standards are based on Western, especially American standards, just like its technical requirements and standards are. When considering human factors, you should naturally take into account the traits of the people involved. However, the Japanese have national traits different from those of Westerners. These two races are sometimes characterized as agricultural people versus stock-farming or maritime people, and in other times, their differences are attributed to the long-term and strong influence of Christianity. For example, Doi points out the following dissimilarities in racial traits in his work, *Amae no Kozo (The Anatomy of Dependence)* [32].

- (i) Westerners are usually aware of the dominance of the individual over the group. They want to consider themselves mentally free from the group and never belonging to the group. On the other hand, the Japanese put the group ahead of the individual. The group is the major support for them. They consider breaking away or being isolated from the group as a loss of self, and feel it an unbearable situation.
- (ii) It is said that for Westerners, a sense of guilt comes from betraying their superego formed in their minds. However, for the Japanese, a sense of guilt is most keenly felt when they realize that their act may constitute a betrayal to the group they belong to.
- (iii) The industriousness of the Japanese is related to the mental state of "*kigasumanai*," or compulsiveness, which strongly manifests itself among the Japanese. The Japanese actually want to depend on others but can hardly do so in reality. As a result, they deny their dependence and fall into the *kigasumanai* state of mind.

Although believing in the dominance of the individual over the group, Westerners may have a certain desire for belongingness, and perhaps some degree of dependence. Despite such similarities in personality, it is clear that the racial traits of

the Japanese are different from those of Westerners. Therefore, applying Western style as is, without considering this difference, can create strain and thus may not be the best approach. For example, this may cause a problem like this regarding quality assurance: Difference in racial traits can lead to difference in the meaning of “acting appropriately” and the “organization designed to facilitate the execution” in the context that “personnel (engineers) in charge should act appropriately in each step of the process, and the organization should be designed to facilitate the execution of this,” a sentence at the beginning of this section. The fact that the Japanese are not the only agricultural people in the world and that there are many people who are not strongly influenced by Christianity indicates what Japan should do. It is significant that, as a world leader in technology, Japan will propose structural standards suited for the traits of agricultural people, especially the concepts of structural standards related to human factors, and eventually take the initiative in constructing systematized concepts that take account of different racial traits.

1.6.3 Development of Structural Standards Based on Probabilistic Approach (Reliability-Based Code)

[1] Background

Essentially, there is uncertainty in everything, including not only the load and material characteristics but also the accuracy of analysis models, fabrication accuracy, installation accuracy, and even examination accuracy. Many of the existing design standards are considered to ensure structural integrity under these uncertainties by limiting the load (or stress, or more commonly, load effect) as shown in Eq. 1.1, using deterministic design factors identified from experience.

This approach is also known as the allowable stress design method (ASD) because it applies design factors only to the strength side to derive so-called allowable stress. However, this approach cannot ensure a consistent quality level due to the unavailability of the influence of individual uncertainties, and therefore may lead to an excessively conservative design. The realization of this drawback has given momentum to efforts to develop standards using a probabilistic approach that takes account of risk involved in design or even the entire life cycle.

[2] Current status

The idea of using the probability of fracture as a structural design method was proposed in Russia in the 1920s. Research in this approach also started in the 1940s in the USA. By the 1990s, institutions such as the American Institute of Steel Construction (AISC) and the American Concrete Institute (ACI) formulated design standards based on the load and resistance factor design method (LRFD). Likewise, the ISO established ISO 2394 “General principles on reliability for structures” [33] in 1973. Discussions have begun in Japan as well. Asada recently proposed the System Based

Code [34, 35], a novel concept that aims to optimize margins throughout the life cycle of equipment.

The concept of LRFD is explained in the following section [3]. This is similar to the limit state design method proposed by ISO 2394. In the limit state design method, failure modes that should be assumed in a given component are identified so that the critical state (a state where the component can no longer fulfill its required functions during the service period) can be defined for each failure mode. This is a design method that assumes that whether or not this critical state is reached is essentially uncertain and intends to address the problem probabilistically. In reality, methods using partial safety factors are being established as standards.

On the other hand, efforts to apply risk-based approaches to maintenance standards are also gaining momentum, as demonstrated by the fact that they have been specified in Section XI of the ASME Boiler and Pressure Vessel Code [36].

[3] LRFD

LRFD requires that the factored (reduced) strength of a component be equal to or more than the linear sum of the factored (magnified) load effects. This can be expressed as

$$\phi R_n \geq \sum_{i=1}^m \gamma_i L_{ni} \quad (1.2)$$

where the symbols are defined as follows:

ϕ : Strength factor.

R_n : Nominal (or design) strength.

γ_i : Load factor of load component i .

L_{ni} : Nominal (or design) value of component i among m pieces of load components.

The strength factor and the load factor are also known as partial safety factors (PSFs) (the term “safety factor” is a little old-fashioned, and it has been replaced by the term “design factor” for reasons explained in Chap. 5). The values of these factors are determined so as to satisfy the target reliability (or quality) level, in consideration of the probabilistic characteristics of materials, structural configuration, and the load obtained from the reliability analysis of the structure.

This chapter explained you moves that will gain worldwide momentum in the near future, including those toward establishing international standards, performance-based quality assurance standards, and probability-based standards. Although these moves may not substantially influence every part of the technical issues discussed from Chap. 2 onward, they still have an impact on these issues as a whole. You are recommended to study by yourself how they will influence.

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Chapter 2

Strength of Materials and Structural Mechanics



This chapter describes the basics of strength of materials required of chief reactor engineers for those readers who studied strength of materials in mechanical engineering courses at university level. References [1, 2] are recommended as introductory level books on the subject for those who never studied it. In recent years, structural strength calculation is usually performed using commercially available numerical analysis programs such as finite element programs. However, without the knowledge of the basics of strength of materials, you cannot set problems (plane stress, plane strain, beams, large deformation buckling, etc.), loading conditions, and boundary conditions and specify data on material properties. Also, it is essential to understand calculation results. For verification purposes, the results are compared with theoretical calculations for ideal cases such as those described in this chapter. If theoretical calculations are not available, experiment is performed to compare the calculation results with the experimental data. Even a computer simulation of structural strength requires the knowledge of strength of materials. At nuclear power plant sites, it is an important skill for professional engineers to be able to quickly estimate by hand calculation the stress, strain, and deformation of the pressure vessel and piping due to an increase in internal pressure, load, and temperature. You should avoid the attitude “I am not an expert on strength of materials, so I know nothing about it.” Instead, you should study it.

This chapter describes in detail the energy method (the principle of minimum potential energy, Castigliano’s theorem, etc.) to help you understand the principle of numerical calculation methods. It also describes how to obtain discrete solutions such as point forces and displacement in simple element meshing by hand calculation. We hope that you will be able to perform analysis using even a large numerical analysis program with an understanding of how it works.

2.1 Stress and Strain

The difference between analytical dynamics in physics and strength of materials and structural mechanics is that, as shown in Fig. 2.1, the former treats an object as a point mass having mass but no dimensions and ignores its deformation while the latter deals with the deformation of an object and the strain energy due to deformation accumulated in it.

The relationship between a force or a load and stress is explained below. A force externally acting on a structure is called an external force or load. The structure is supported and fixed at some points. Therefore, when an external force is applied to the structure, it is subjected to a repulsive force against the external force at the support. Let's take as an example a case where a force is applied to a round bar at one end. If the bar is not supported, it moves. If supported, the bar is subjected to a force of the opposite direction (called a support reaction) at this support. It deforms due to the external force and the reaction from both sides. This deformation occurs over the entire length of the bar. This is because the external force and the reaction at the support are transmitted through the bar. The force generated in the structure by the external force is called an internal force. The internal force per unit cross-sectional area at the point subject to an integrity evaluation is called stress. The purpose of strength of materials is to evaluate support reactions to external forces and the resulting stresses. The reason why the evaluation is performed per unit cross-sectional area is that you would want to evaluate forces and stresses using dimensionless quantities independent of the size of the structure. Loads include thermal loads. A thermal load is not an externally applied load, but is the load resulting from the structure being constrained to deform (this constrained state is different from the supported state mentioned above). The failure mode of a component depends on whether the load is externally applied or induced by constraint. Therefore, the concept of classification of loads and stresses is used in structural standards.

In strength of materials and structural mechanics, parameters in several directions must be considered to treat an object of finite size and describe displacement and deformation. As illustrated in Fig. 2.2, an object of finite size deforms and has displacement components in the direction of the load, as well as other directions.

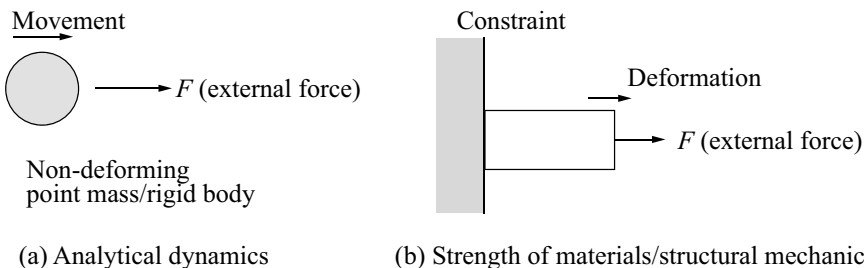
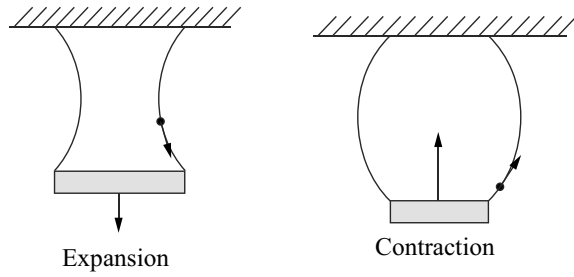


Fig. 2.1 Movement and deformation by an external force in analytical dynamics and strength of materials/structural mechanics

Fig. 2.2 Deformation of an object of finite size in directions other than the direction of the load



We assume that in a Cartesian coordinate system, the load and the deformation have three components (x , y , z) and there is a linear relationship between the load and the deformation. Then, nine factors are required for the relationship. An accurate hand calculation of all these factors is difficult.

In strength of materials, given a good understanding of the characteristics of the system, ignorable quantities can be omitted, reducing unknown quantities and allowing hand calculation. This chapter deals mainly with only four systems as shown in Fig. 2.3. In these systems, for example, the number of parameters, such as the load applied, is reduced to two or less, and the number of quantities, such as stress, strain, and displacement, to be obtained is reduced to two or less to allow hand calculation. This allows you to understand the theory and the concept by moving your hand.

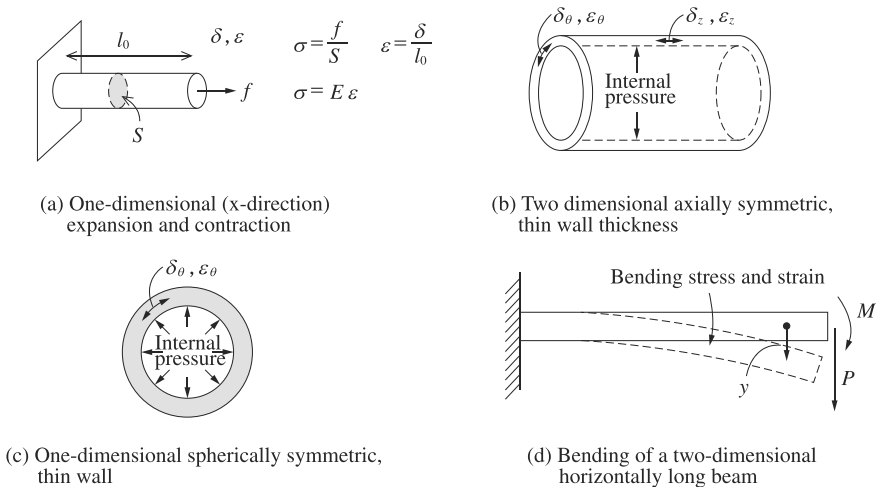
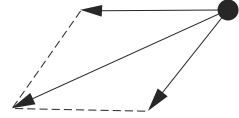
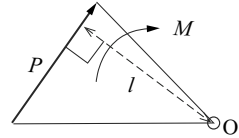


Fig. 2.3 Systems dealt with in this chapter and degrees of freedom

Fig. 2.4 Force (vector)**Fig. 2.5** Decomposition and composition of forces**Fig. 2.6** Moment

2.1.1 Forces

Forces include gravitational, wind, and seismic forces, which cause a stationary object to move or a moving object to change its direction of movement or acceleration.

Notation of force: magnitude, direction, and point of application (Fig. 2.4).

Decomposition and composition of force (Fig. 2.5).

2.1.2 Moments

The moment of force P at point O is the product of the distance to the perpendicular distance l from point O times the force P . In Fig. 2.6, let M be a moment.

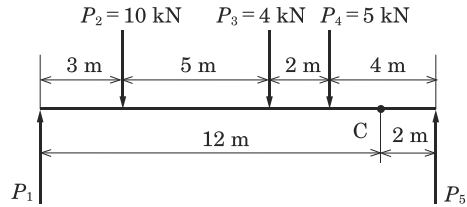
$$M = P \cdot l \quad (2.1)$$

2.1.3 Equilibrium of Forces

If when two or more forces act on one point, the resultant force is zero, and the point is stationary, the forces acting on that point are in equilibrium.

When two or more forces (a set of forces) acting on a structure are in static equilibrium, both the resultant force and moment of the set of forces must be zero.

$$\sum_i P_i = 0 \quad (2.2)$$

Fig. 2.7 Equilibrium of forces

$$\sum_i M_i = \sum_i P_i l_i = 0 \quad (2.3)$$

The five forces in Fig. 2.7 are in equilibrium.

- (1) Calculate forces P_1 and P_2 .
- (2) Calculate the moment acting on point C.

2.1.4 Stress

When an external force (load) acts on an object, a resisting force (internal force) is generated in the material composing the object. The internal force per unit cross-sectional area is called stress.

There are the following types of stress:

- (a) Tensile stress.
- (b) Compressive stress.
- (c) Shear stress.

The solid and dashed lines in (a) to (c) show stress before deformation and after deformation, respectively (Fig. 2.8).

Normal stress: $\sigma = P/A$

Tensile stress (+).....**Figure 2.8 (a)** (2.4)

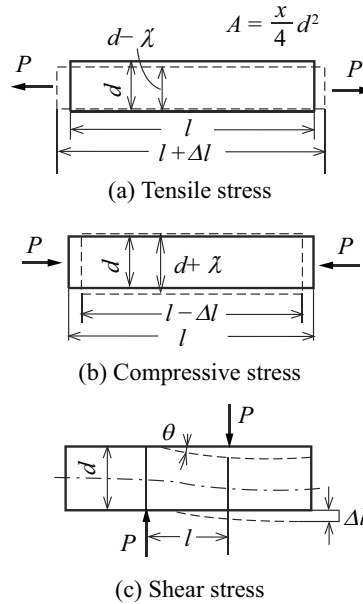
Compressive stress (-).....**Figure 2.8 (b)**

Shear stress:.....**Figure 2.8 (c)**

Unit of stress : Pa (=N/m²) (2.5)

(Normally, MPa is used.)

A: Cross-sectional area of the member.

Fig. 2.8 Types of stress

The solid and dashed lines in (a) to (c) show stress before deformation and after deformation, respectively."

2.1.5 Strain and Compatibility Conditions

To describe the deformation of an object subjected to a load, the amount of deformation per unit length of the object is used. This is called strain.

There are the following types of strain:

$$\text{Normal strain: } \epsilon = \Delta l / l \quad (2.6)$$

Tensile strain (+).....Figure 2.8 (a)

$$\text{Compressive strain (-).....Figure 2.8 (b)} \quad (2.7)$$

$$\text{Shear strain: } \gamma = \Delta l / l = \theta \quad \text{Figure 2.8 (c)}$$

$$\text{Transverse strain: } \epsilon' = \lambda' / d \quad \text{Figures 2.8 (a), (b)} \quad (2.8)$$

$$\text{Volumetric strain: } \epsilon_v = \Delta V / V \quad (2.9)$$

(Volumetric strain ϵ_v when an object is subjected to a load and its volume changes $V \rightarrow V + \Delta V$).

2.1.6 Hooke's Law and Young's Modulus

When a load is applied to an object, it deforms. When the load is removed, the deformation is completely removed. This tendency is called elasticity. The maximum stress up to which this tendency is retained is called the elastic limit. Beyond the elastic limit, part of the deformation remains in the object even after the load is removed. This tendency is called plasticity, and this deformation is called plastic deformation.

Stress is proportional to strain within the elastic limit. This relationship is called Hooke's Law, and the proportionality constant for the stress–strain relationship is called elastic modulus.

$$\sigma = E\varepsilon \quad (2.10)$$

E : Longitudinal elastic modulus (normally called Young's modulus)

$$\tau = G\gamma \quad (2.11)$$

G : Transverse elastic modulus

$$p = K\varepsilon_v \quad (2.12)$$

K : Volumetric elastic modulus

The relationships among E , G , and K are given by

$$G = \frac{E}{2(1+\nu)} \quad (2.13)$$

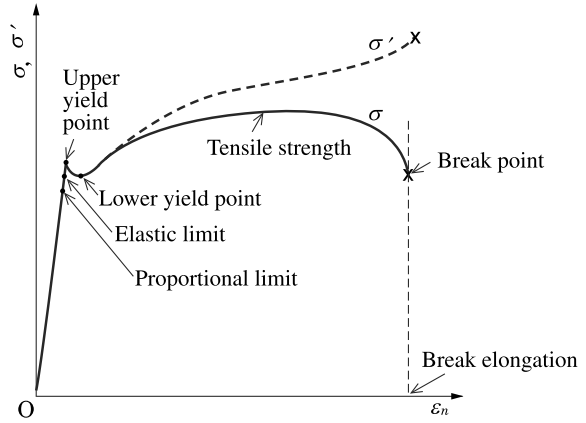
$$K = \frac{E}{3(1-2\nu)} \quad (2.14)$$

Here, ν is the Poisson's ratio (defined as the negative of the ratio of the transverse strain to the longitudinal strain).

$$\nu = -\frac{\varepsilon'}{\varepsilon}$$

2.1.7 Stress–Strain Relationship and Constitutive Equations

When designing a component or a structure, the material must be selected according to the loading and ambient conditions. To select the material, material testing needs to be performed to investigate its elastic and plastic behavior.

Fig. 2.9 Stress–strain curve

The most important and basic test to investigate the mechanical properties of material is a tensile test. Normally, a tensile load is applied to a round bar or a flat plate as a test specimen to obtain the load-elongation relationship of the material. A stress–strain curve is obtained by dividing the load by the original cross-sectional area (A) of the test specimen and the elongation by the gage length (l) and changing the scale of the coordinates. The thus-obtained stress–strain relationship is called a constitutive equation of material.

As shown in Fig. 2.9, in a small strain region, the stress–strain relationship is linear. The threshold stress is called the proportional limit. The limit below which when the load is removed, permanent strain (below 0.01 to 0.03% strain) does not occur, is called the elastic limit. The point where the maximum value beyond the linear elastic region is reached is called the upper yield stress. The minimum point after the maximum point is called the lower yield stress. The point where the maximum load is applied in the nonlinear region is called the tensile strength. The point where the specimen breaks as the strain further increases and the load decreases is called the break point.

Nominal stress:

$$\sigma = P/A \quad (2.15)$$

True stress:

$$\sigma' = P/A' \quad (2.16)$$

(A' is the actual cross-sectional area taking into account the reduction of the cross-sectional area of the specimen.)

Nominal strain:

$$\epsilon_n = (l' - l)/l \quad (2.17)$$

True strain:

$$\varepsilon_l = \int_l^r \frac{dl}{l} = \ln\left(\frac{l'}{l}\right) = \ln(1 + \varepsilon_n), \quad (2.18)$$

where ε_l is also called natural strain or logarithmic strain.

In this chapter, unless otherwise stated, material is treated as an elastic body that follows Hooke's law.

2.1.8 A Bar Subjected to a Tensile or Compressive Load

We calculate elongation Δl when a tensile load P is applied to a bar with a length l and a uniform cross-sectional area A . Let E be Young's modulus. Then, stress σ and strain ε in the bar are given by

$$\sigma = P/A, \quad \varepsilon = \Delta l/l$$

From Hooke's law, we have

$$\begin{aligned} \sigma &= E\varepsilon \\ \therefore P/A &= E\Delta l/l \\ \therefore \Delta l &= Pl/AE \end{aligned}$$

[Example Problem] A bar with a uniform cross-sectional area is fixed at the upper end and subjected to a downward tensile load P . Let ρ be the specific gravity, A be the cross-sectional area, and l be the length. How can we describe the stress σ and the elongation Δl of the entire bar due to P and the dead load?

[Example Answer] As shown in Fig. 2.10, at the section X-X at a distance x from the lower end, the bar is subjected to the sum of the dead load of the portion below section X-X and the tensile load P , resulting in a tensile stress given by

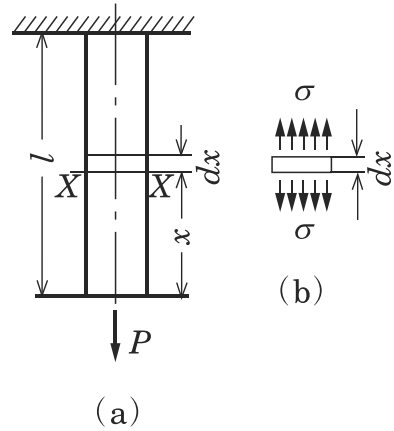
$$\sigma = \frac{P + \rho Ax}{A} = \frac{P}{A} + \rho x$$

The stress increases in proportion to the distance x from the lower end of the bar and reaches the maximum at the upper end ($x = l$).

$$\sigma_{\max} = \frac{P}{A} + \rho l$$

We focus on the elongation of the bar in the region with a length dx , adjacent to section X-X. In this small region, the bar is subject to elongation due to the above stress σ , which is given by

Fig. 2.10 Description of stress taking into account the dead load



$$d\Delta l = \epsilon dx = \frac{\sigma}{E} dx = \frac{1}{E} \left(\frac{P}{A} + \rho x \right) dx$$

The elongation of the entire bar can be obtained by integrating this equation.

$$\Delta l = \int d\Delta l = \frac{1}{E} \int_0^l \left(\frac{P}{A} + \rho x \right) dx = \frac{1}{AE} \left(Pl + \frac{l}{2} Al\rho \right) = \frac{l}{AE} \left(P + \frac{W}{2} \right)$$

where $W = Al\rho$ is the total weight of the bar.

Therefore, when the dead load of the bar is considered, the dead P plus a half of the live load can be considered as the total load to calculate the elongation of the bar.

2.1.9 Residual Stress

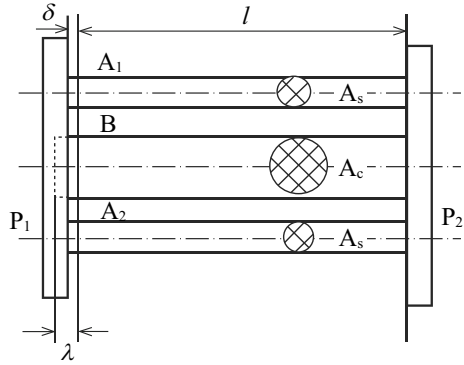
In some cases, stress is present in a component or a structure with no load applied. This is called initial stress or residual stress.

For example, in Fig. 2.11, a copper bar B with a length $(l + \lambda)$ slightly longer than the plate spacing l is forced into the center of a frame consisting of two rigid plates P1 and P2 and two steel bars A_1 and A_2 . As a result, the spacing l becomes $(l + \delta)$, causing the elongation δ of the steel bar and the contraction $(\lambda - \delta)$ of the copper bar.

Consequently, the tensile or compressive forces generated in each bar must be in balance with each other.

$$2A_s E_s \frac{\delta}{l} = A_c E_c \frac{\lambda - \delta}{l} \quad (2.19)$$

Fig. 2.11 A residual stress system



where E_s and E_c are the Young's modulus of steel and copper, respectively, and A_s and A_c are the cross-sectional area of the steel bar and the copper bar, respectively. For the copper bar, $l + \lambda \approx \lambda$ due to $l \gg \lambda$. Solving this equation for δ , we obtain

$$\delta = \frac{A_c E_c \lambda}{2A_s E_s + A_c E_c} \quad (2.20)$$

Hence, σ_s and σ_c in the steel and copper bars are given by

$$\sigma_s = E_s \varepsilon_s = E_s \frac{\delta}{l} = \frac{A_c E_c E_s}{2A_s E_s + A_c E_c} \cdot \frac{\lambda}{l} \quad (2.21)$$

$$\sigma_c = E_c \varepsilon_c = E_c \frac{\lambda - \delta}{l} = \frac{2A_s E_s E_c}{2A_s E_s + A_c E_c} \cdot \frac{\lambda}{l} \quad (2.22)$$

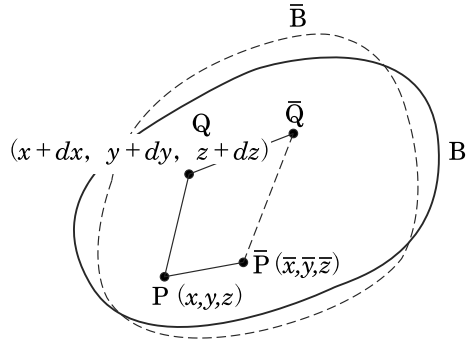
2.1.10 Basic Equations to Define a Structural Mechanics Problem

[1] Compatibility Conditions

If an object B is subjected to an external force and deformed into B, the line segment PQ moves to PQ (Fig. 2.12). If the coordinate system before deformation is used to describe points in the object and the point P (x, y, z) is displaced to P ($x + u, y + v, z + w$), (u, v, w) is called displacement components. If the deformation is small, the strains at this point are given by.

$$\left. \begin{aligned} \varepsilon_x &= \partial u / \partial x, \quad \varepsilon_y = \partial v / \partial y, \quad \varepsilon_z = \partial w / \partial z \\ \gamma_{xy} &= \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x}, \quad \gamma_{yz} = \frac{\partial v}{\partial z} + \frac{\partial w}{\partial y}, \quad \gamma_{zx} = \frac{\partial w}{\partial x} + \frac{\partial u}{\partial z} \end{aligned} \right\} \quad (2.23)$$

Fig. 2.12 Deformation of an object

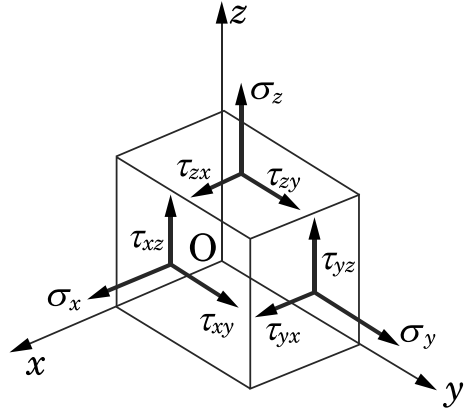


where ε_x , ε_y , and ε_z are the normal strains in the x , y , and z -directions, and γ_{xy} , γ_{yz} , and γ_{zx} are the shear strains in the x_y , y_z , and z_x planes. These are called strain components, and the deformation of an object is defined by the strain components.

As shown in Eq. (2.23), the six strain components are given by the derivatives of the three displacement components. Therefore, they are not independent of each other and must meet a certain relationship. The following equations are obtained by eliminating the displacement components in Eq. (2.23).

$$\left. \begin{aligned} \frac{\partial^2 \varepsilon_x}{\partial y^2} + \frac{\partial^2 \varepsilon_y}{\partial x^2} &= \frac{\partial^2 \gamma_{xy}}{\partial x \partial y} \\ \frac{\partial^2 \varepsilon_y}{\partial z^2} + \frac{\partial^2 \varepsilon_z}{\partial y^2} &= \frac{\partial^2 \gamma_{yz}}{\partial y \partial z} \\ \frac{\partial^2 \varepsilon_z}{\partial x^2} + \frac{\partial^2 \varepsilon_x}{\partial z^2} &= \frac{\partial^2 \gamma_{zx}}{\partial z \partial x} \\ 2 \frac{\partial^2 \varepsilon_x}{\partial y \partial z} &= \frac{\partial}{\partial x} \left(-\frac{\partial \gamma_{yz}}{\partial x} + \frac{\partial \gamma_{zx}}{\partial y} + \frac{\partial \gamma_{xy}}{\partial z} \right) \\ 2 \frac{\partial^2 \varepsilon_y}{\partial z \partial x} &= \frac{\partial}{\partial y} \left(\frac{\partial \gamma_{yz}}{\partial x} - \frac{\partial \gamma_{zx}}{\partial y} + \frac{\partial \gamma_{xy}}{\partial z} \right) \\ 2 \frac{\partial^2 \varepsilon_z}{\partial x \partial y} &= \frac{\partial}{\partial z} \left(\frac{\partial \gamma_{yz}}{\partial x} + \frac{\partial \gamma_{zx}}{\partial y} - \frac{\partial \gamma_{xy}}{\partial z} \right) \end{aligned} \right\} \quad (2.24)$$

This set of equations is geometrically defined and called a compatibility equation. Displacement is uniquely determined only if a strain component is given to meet the compatibility equation.

Fig. 2.13 Axial stresses**[2] Constitutive Equation.**

The equation to describe a stress–strain relationship that determines the properties of a material is called a constitutive equation.

In the Cartesian coordinate system shown in Fig. 2.13, the three-dimensional stress–strain relationship for an elastic body is given by

$$\left. \begin{aligned} \epsilon_x &= \frac{1}{E} \{ \sigma_x - \nu(\sigma_y + \sigma_z) \} \\ \epsilon_y &= \frac{1}{E} \{ \sigma_y - \nu(\sigma_z + \sigma_x) \} \\ \epsilon_z &= \frac{1}{E} \{ \sigma_z - \nu(\sigma_y + \sigma_x) \} \\ \gamma_{xy} &= \frac{1}{G} \tau_{xy}, \quad \gamma_{yz} = \frac{1}{G} \tau_{yz}, \quad \gamma_{zx} = \frac{1}{G} \tau_{zx} \end{aligned} \right\} \quad (2.25)$$

The above set of equations holds in a Cartesian coordinate system, as well as in the principal stress state (1, 2, 3) and in a cylindrical coordinate system (r, θ, z).

Normally, material testing is required to determine the constitutive equation.

[3] Equilibrium Conditions

The stress at any point in an object is described by the stress components in the periphery of a small hexahedron including that point. These components must be in equilibrium with body forces (gravity, centrifugal force, inertia, etc.). This equilibrium of forces is given by the equations of equilibrium

$$\left. \begin{aligned} \frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} + \frac{\partial \tau_{xz}}{\partial z} + X &= 0 \\ \frac{\partial \tau_{yx}}{\partial x} + \frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{yz}}{\partial z} + Y &= 0 \\ \frac{\partial \tau_{zx}}{\partial x} + \frac{\partial \tau_{zy}}{\partial y} + \frac{\partial \sigma_z}{\partial z} + Z &= 0 \end{aligned} \right\} \quad (2.26)$$

where X , Y , and Z are respectively the x , y , z components of the body force per unit volume.

2.2 Thermal Stress

If there are temperature differences across a component or a structure, or if there is a difference in thermal expansion or contraction due to different materials used, each material is constrained from free deformation, resulting in a condition similar to being subjected to an external force and causing stress. This stress is called thermal stress.

Assume that, as shown in Fig. 2.14, a bar AB with a length l is fixed to rigid walls at both ends at a temperature T_1 ($^{\circ}\text{C}$) and heated to T_2 ($^{\circ}\text{C}$). Let α be the coefficient of linear expansion of the bar. The bar is constrained from expansion by the rigid walls, which otherwise would expand by $\alpha l (T_2 - T_1)$, and is subjected to the same effect as being subjected to the axial compressive strain given by

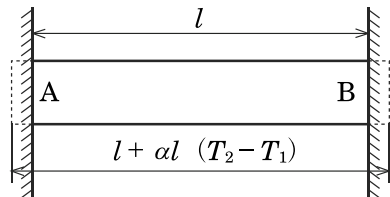
$$\varepsilon = \frac{\alpha l (T_2 - T_1)}{l} = \alpha (T_2 - T_1) \quad (2.27)$$

As a result, the compressive stress given by the following equation is generated:

$$\sigma = E\varepsilon = E\alpha(T_2 - T_1) \quad (2.28)$$

[Example Problem] As shown in Fig. 2.15, a bar B is running through in the center of a pipe A, and the pipe and the bar are fixed to rigid plates at both ends. Calculate σ_a and σ_b when the temperature of the pipe and the bar increases by ΔT (=

Fig. 2.14 A thermal stress system



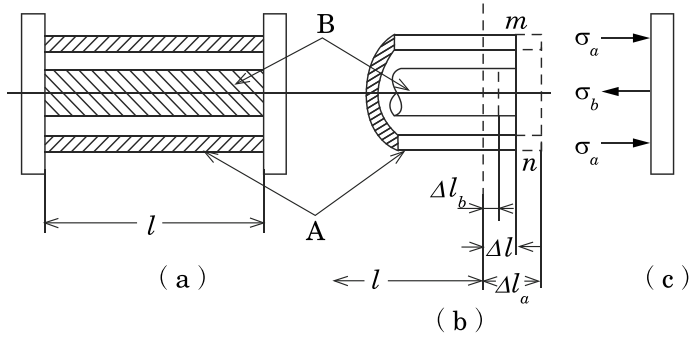


Fig. 2.15 A system of a pipe and a bar made of different materials

$T_2 - T_1$). Here, let E_a and E_b be the Young's modulus of the pipe A and the bar B, respectively, A_a and A_b be the cross-sectional area, and α_a and α_b be the coefficient of linear expansion.

[Example Answer] We solve the compatibility, constitutive, and force equilibrium equations described in Sect. 2.1.10 in this order.

With no constraints, the pipe A and the bar B would expand by Δl_a and Δl_b , respectively. However, due to the constraints at both ends, both the pipe and the bar expand only by Δl and remain within the m-n curve.

The pipe A is subjected to compression equivalent to contraction $(\Delta l_a - \Delta l)$ and the bar B is subjected to tension equivalent to expansion $(\Delta l - \Delta l_b)$. The strain in the pipe and the bar is given by

$$\epsilon_a = \frac{\Delta l_a - \Delta l}{l + \Delta l_a} \cong \frac{\Delta l_a - \Delta l}{l}$$

$$\epsilon_b = \frac{\Delta l - \Delta l_b}{l + \Delta l_b} \cong \frac{\Delta l - \Delta l_b}{l}$$

The following thermal stress is generated in the pipe and the bar:

$$\sigma_a = E_a \epsilon_a = E_a \frac{\Delta l_a - \Delta l}{l} \quad (\text{Compressive stress})$$

$$\sigma_b = E_b \epsilon_b = E_b \frac{\Delta l - \Delta l_b}{l} \quad (\text{Tensile stress})$$

These stresses act on the rigid plates on both ends and must be balanced with each other. Therefore,

$$\sigma_a A_a = \sigma_b A_b$$

Since the thermal expansion of the pipe and the bar is $\Delta l_a = l\alpha_a\Delta T$ and $\Delta l_b = l\alpha_b\Delta T$, respectively, we obtain

$$E_a \frac{l\alpha_a\Delta T - \Delta l}{l} A_a = E_b \frac{\Delta l - l\alpha_b\Delta T}{l} A_b$$

Solving this equation for Δl , we obtain

$$\Delta l = \frac{E_a\alpha_a A_a + E_b\alpha_b A_b}{E_a A_a + E_b A_b} \Delta T l$$

Then, the stress in the pipe and the bar is given by

$$\sigma_a = \frac{E_a E_b A_b \Delta T (\alpha_a - \alpha_b)}{E_a A_a + E_b A_b}$$

$$\sigma_b = \frac{E_a E_b A_a \Delta T (\alpha_a - \alpha_b)}{E_a A_a + E_b A_b}$$

2.3 Beams

Beam: A bar supported at more than one point and subjected to a load having a force component normal to its axis is called a beam (Fig. 2.16).

Support point: a point to support the beam.....A, B

(Movable support, hinged support, fixed support).

Load: an external force acting on the beam.....H, P_1 , P_2 .

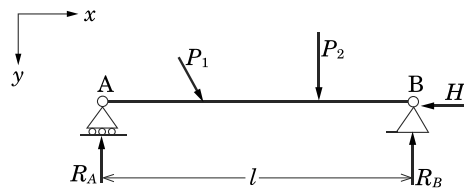
(Dead weight of the beam, weight supported by the beam, wind force, etc.)

Movable load, fixed load, concentrated load, distributed load.

Support reaction: a force induced at the support to balance the load.

Span: the support-to-support distance.....l

Fig. 2.16 A beam system



2.3.1 Shear Forces and Bending Moments

We consider a case where a concentrated load P is applied to a simply supported beam with a span l at point C between points A and B (Fig. 2.17a).

We assume that the beam is divided into left and right parts at section X-X at a distance x from point A and consider conditions for force equilibrium on the left side of the beam. Then, a vertical force F_1 is to balance R_a as well as a moment M_1 to balance the moment induced by R_a and F_1 act on the beam (Fig. 2.17b).

The force F_1 acts to shear the cross section of the beam and therefore is called a shear force. The moment M_1 acts to bend the axis of the beam in the symmetric plane and therefore is called a bending moment.

Similarly, we consider conditions for force equilibrium on the right side and assume that a shear force F_2 and a bending moment M_2 act on the beam.

$$F_1 = R_a, \quad F_2 = P - R_b$$

$$M_1 = R_a x, \quad M_2 = R_b(l-x) - P(l_1-x)$$

From the equilibrium of forces and moments, the following equations are obtained:

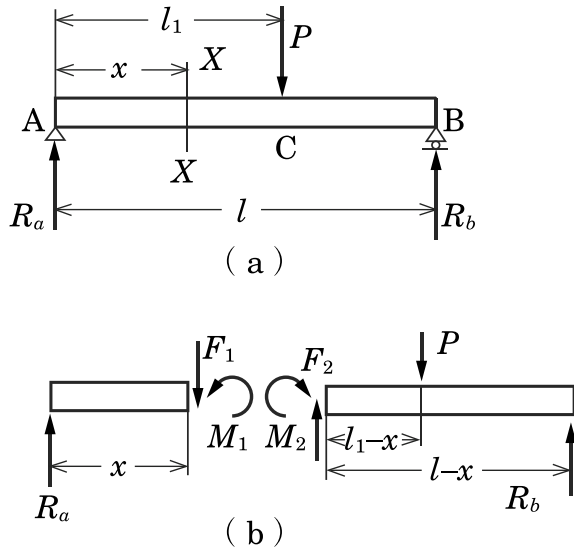
$$\text{From } F_1 = F_2, \quad R_a + R_b = P$$

$$\text{From } M_1 = M_2, \quad R_a l + P(l - l_1) = 0$$

$$\therefore R_a = P(l - l_1)/l, \quad R_b = Pl_1/l$$

Shear force F

Fig. 2.17 Shear forces and moments



Between points A and C: $F = P(l-l_1)/l$

Between points C and B: $F = -Pl_1/l$

(Here, the plus/minus sign is a convention.)

Bending moment $M(x)$:

Between points A and C: $M(x) = R_ax = Px(l-l_1)/l$ (monotonically increasing).

Between points C and B: $M(x) = R_ax - P(x-l_1) = Pl_1(l-x)/l$ (monotonically decreasing).

Therefore, the maximum bending moment $M_{\max}$ occurs at the load-application point C.

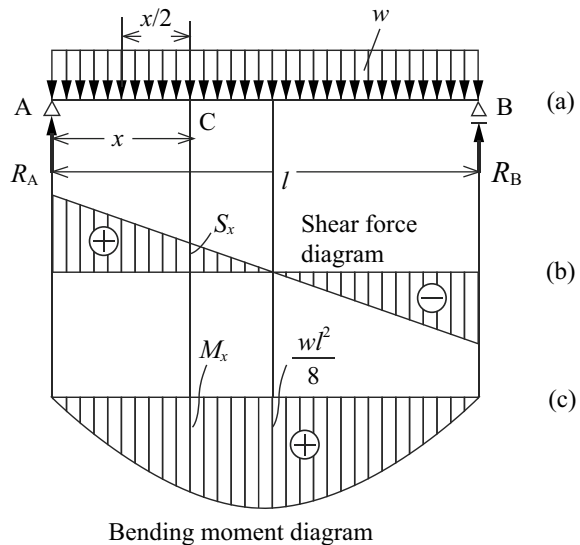
$$M_{\max} = M(l_1) = Pl_1(l-l_1)/l$$

2.3.2 Simply Supported Beam

We consider a simply supported beam subjected to a uniformly distributed load w (see Fig. 2.18a).

We calculate reactions R_A and R_B at both supports from the force equilibrium conditions.

Fig. 2.18 A simply supported beam



$$R_A + R_B = wl$$

$$R_A \cdot l - \int_0^l wx dx = R_A \cdot l - wl^2/2 = 0$$

$$\therefore R_A = wl/2, \quad R_B = wl/2$$

From the equilibrium of forces in the beam left of point C, $S(x)$ is

$$S(x) = R_A - wx = w(l/2 - x)$$

The bending moment $M(x)$ is

$$M(x) = R_A \cdot x - \int_0^x w dx'(x - x') = wx/2 - wx^2/2 = wx(l - x)/2$$

Then, the maximum bending moment $M_{\max}$ occurs at $x = l/2$ and is given by

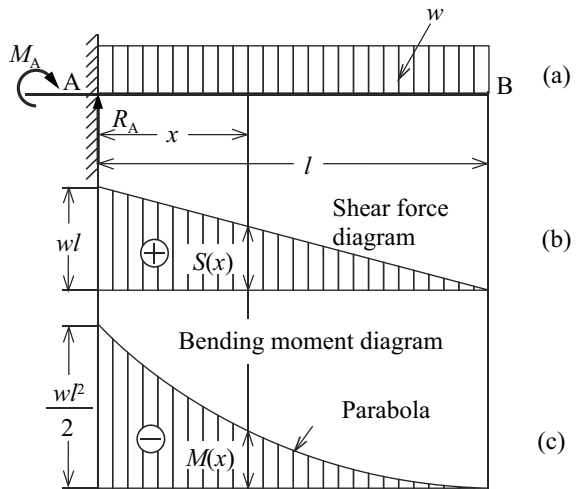
$$M_{\max} = M(l/2) = wl^2/8$$

2.3.3 Cantilever Beam

We consider a cantilevered beam subjected to a uniformly distributed load w (see Fig. 2.19a).

$$\text{Support reaction: } R_A = wl, \quad M_A = -\int_0^l wx dx = -wl^2/2$$

Fig. 2.19 A cantilever beam



Shear force: $S(x) = w(l-x)$

Bending moment: $M(x) = -w(l-x)^2/2$

Maximum bending moment (at point A): $M_{\max} = M(0) = -wl^2/2$

2.3.4 Stresses in the Beam

As shown in Fig. 2.20, the x -axis is taken in the neutral plane of the beam, the y -axis is taken in the load-application plane normal to the neutral plane, and z -axis is taken normal to the y -axis. Let M be the bending moment acting on the beam and r be the radius of curvature of the beam bent into an arc by the moment.

For the y -direction, the strain ε and the stress σ at m_1n_1 are given by

$$\varepsilon = \frac{y}{r}, \quad \sigma = E\varepsilon = E \frac{y}{r}$$

The bending moment M is given by

$$M = \int_A \sigma \cdot y dA = \frac{E}{r} \int_A y^2 dA \quad (2.29)$$

Here, let the moment of inertia in the neutral axis be $(\int_A y^2 dA = I)$. Then, the relationship between the radius of curvature of the beam and the bending moment is given by

Fig. 2.20 Stress in the beam

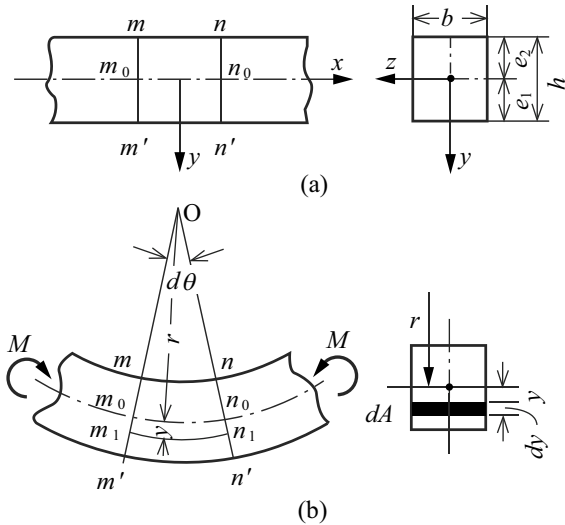
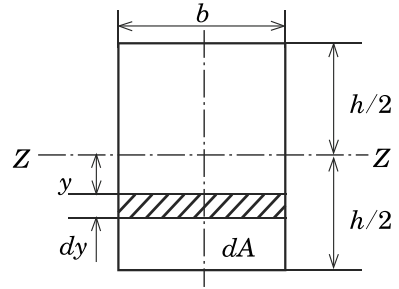


Fig. 2.21 A system for calculating the moment of inertia for a rectangular cross-section



$$\frac{1}{r} = \frac{M}{EI} \quad (2.30)$$

The relationship between the bending stress σ_M and the bending moment M is given by

$$\sigma_M = Ey \cdot \frac{1}{r} = Ey \frac{M}{EI} = \frac{M}{I} y \quad (2.31)$$

The maximum bending stress $\sigma_{M_{\max}}$ occurs at $y = h/2$. Hence,

$$\sigma_{M_{\max}} = \frac{M}{I} \cdot \frac{h}{2} = \frac{M}{Z} \quad (2.32)$$

where Z is called section modulus ($= I/(h/2)$).

For reference purposes, how to calculate the moment of inertia for a rectangular cross section ($b \times h$) is described below (Fig. 2.21).

$$I = \int_A y^2 dA = \int_{-h/2}^{h/2} y^2 b dy = \frac{bh^3}{12} \quad (\text{See Figure 2.21}) \quad (2.33)$$

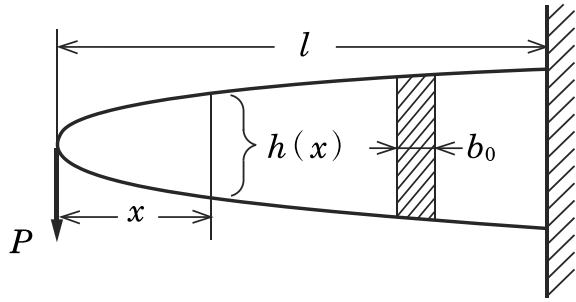
The moment of inertia for a hollow cylindrical cross section with an outer diameter d_2 and an inner diameter d_1 is given by

$$I = \frac{\pi}{64} (d_2^4 - d_1^4) \quad (2.34)$$

2.3.5 A Uniform Strength Beam

As shown in Fig. 2.22, we consider a case where a concentrated load P is applied to the end of a cantilever beam with a uniform width b_0 and a varying height h . How

Fig. 2.22 A uniform strength beam



do you change h to make the maximum bending stress constant along the length of the beam?

The bending moment $M(x)$ at a distance x from the load-application end of the beam is

$$M(x) = -Px$$

The section modulus of the rectangular cross section Z_x is

$$Z(x) = \frac{b_0 h(x)^3}{12} \bigg/ \frac{h(x)}{2} = \frac{b_0 h(x)^2}{6}$$

Hence, the maximum bending moment $\sigma_{\max}$ at any cross section is

$$\sigma_{\max} = \left| \frac{M(x)}{Z(x)} \right| = \frac{6Px}{b_0 h(x)^2}$$

For $\sigma_{\max}$ to be constant,

$$x/h(x)^2 = \text{constant or } h(x) = C\sqrt{x}$$

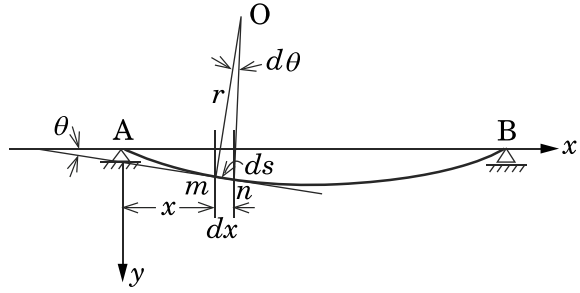
For example, if h is given by h_0 for $x = l$, we have

$$\frac{x}{h(x)^2} = \frac{l}{h_0^2} \quad \therefore \quad h(x) = h_0 \sqrt{\frac{x}{l}}$$

2.3.6 Beam Deflection

Deflection curve of a beam: Let y be deflection and M be a bending moment (see Fig. 2.23).

Fig. 2.23 Deflection of a beam



$$ds = -r \cdot d\theta, \quad dx = ds \cdot \cos\theta$$

The curvature of the beam is given by

$$\frac{1}{r} = -\frac{d\theta}{ds} = -\frac{d\theta}{dx} \cdot \frac{dx}{ds}$$

We have

$$\frac{dy}{dx} = \tan\theta, \quad \theta = \tan^{-1}\left(\frac{dy}{dx}\right)$$

From these equations, we obtain

$$\frac{d\theta}{dx} = \frac{d}{dx} \left\{ \tan^{-1}\left(\frac{dy}{dx}\right) \right\} = \frac{1}{1 + \left(\frac{dy}{dx}\right)^2} \cdot \frac{d^2y}{dx^2}$$

We have

$$\frac{dx}{ds} = \cos\theta = \frac{1}{\sqrt{1 + \tan^2\theta}} = \frac{1}{\sqrt{1 + \left(\frac{dy}{dx}\right)^2}}$$

Then, we obtain

Then, we obtain

$$\frac{1}{r} = -\frac{d^2y/dx^2}{1 + (dy/dx)^2} \cdot \frac{1}{\sqrt{1 + (dy/dx)^2}} = -\frac{d^2y/dx^2}{\{1 + (dy/dx)^2\}^{3/2}}$$

If $\tan\theta = dy/dx < 1$ in this equation, we have

$$\frac{1}{r} \approx -\frac{d^2 y}{dx^2}$$

Using the relationship in Eq. (2.30), we obtain

$$EI \frac{d^2 y}{dx^2} = -M \quad (2.35)$$

This is the basic differential equation to obtain $y = (f x)$ for a deflection curve and called the differential equation of deflection. The deflection angle θ ($\equiv \tan \theta = dy/dx$) and the deflection y can be obtained by integrating this equation. The constant of integration introduced is obtained from the boundary conditions such as the deflection and the deflection angle.

For example, at a hinged support, the deflection equals 0 and the moment equals 0. At a fixed support, the deflection equals 0 and the deflection angle equals 0.

Similarly, we obtain the following equations for a concentrated load P and a distributed load w :

$$EI \frac{d^3 y}{dx^3} = -P \quad (2.36)$$

$$EI \frac{d^4 y}{dx^4} = -w \quad (2.37)$$

[Example Problem] Calculate a deflection curve of a cantilever beam (E , I) subjected to a uniformly distributed load w over the length l , as shown in Fig. 2.24.

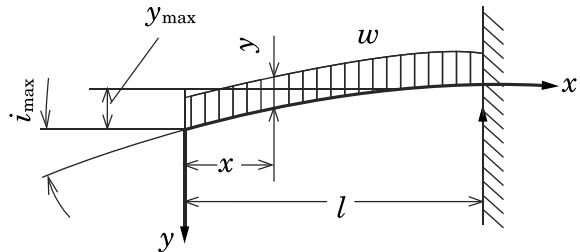
[Hint] The bending moment M at a distance x from the end of the beam:

$$M = -\frac{w}{2} x^2$$

Boundary conditions: for $x = l$, the deflection angle $dy/dx = 0$ and the deflection $y = 0$.

[Example Answer] The differential equation of the deflection curve is given by

Fig. 2.24 A cantilever beam subjected to a uniformly distributed load



$$EI \frac{d^2 y}{dx^2} = \frac{w}{2} x^2$$

The following equations are obtained by integrating this equation:

$$EI \frac{dy}{dx} = \frac{w}{6} x^3 + C_1$$

$$EI y = \frac{w}{24} x^4 + C_1 x + C_2$$

C_1 and C_2 are determined so that the boundary conditions are met. Then, we have

$$C_1 = -\frac{w}{6} l^3, \quad C_2 = \frac{w}{8} l^4$$

$$\begin{aligned} \therefore \frac{dy}{dx} &= \frac{w}{6EI} (x^3 - l^3) \\ y &= \frac{w}{24EI} (x^4 - 4lx + 3l^4) \end{aligned}$$

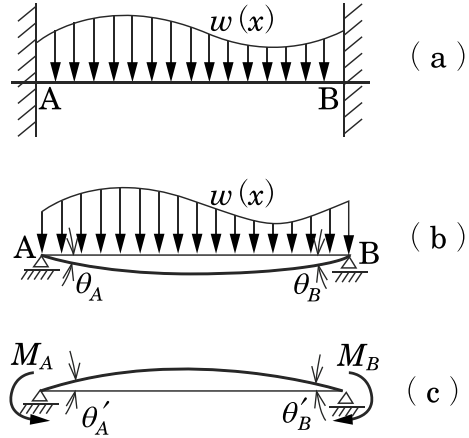
Therefore, the maximum deflection angle ($i_{\max}$) and the maximum deflection ($y_{\max}$) occur at the free end ($x = 0$).

$$\begin{aligned} \left(\frac{dy}{dx} \right)_{\max} &= i_{\max} = -\frac{wl^3}{6EI} \\ y_{\max} &= \frac{wl^4}{8EI} \end{aligned}$$

2.3.7 *Statically Indeterminate Beam*

A beam for which a solution cannot be obtained from an equation of static equilibrium only is called a statically indeterminate beam (see Fig. 2.25).

Fig. 2.25 A statically indeterminate beam



There are two methods for solving the problem of a statically indeterminate beam.

(1) Integration method

Integrating $EI \frac{d^4 y}{dx^4} = w(x)$ four times.

Boundary conditions: for $x = 0, l$ (beam length), $y = 0$ and $dy/dx = 0$.

(2) Superposition method

The statically indeterminate beam shown in Fig. 2.25a is treated as a combination of simply supported beams (b) and (c). The relationship between the deflection angles at both ends is given by

$$\theta_A + \theta'_A = 0, \quad \theta_B + \theta'_B = 0$$

[Example Problem] Calculate the deflection y of the simply supported beam, with a length l , subjected to a concentration load P as shown in Fig. 2.26.

(1) Integration method

$$M = M_A + R_A x \quad (0 \leq x \leq a)$$

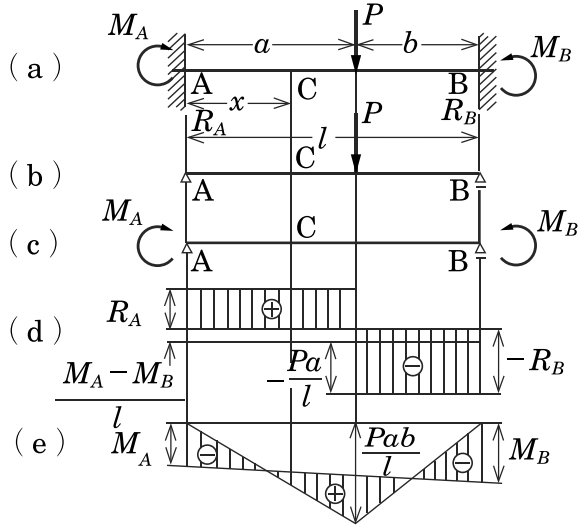
$$M = M_A + R_A x - P(x - a) \quad (a \leq x \leq l)$$

Hence, if $0 \leq x \leq a$, we have

$$EI \frac{d^2 y}{dx^2} = -M_A - R_A x$$

If $a \leq x \leq l$, we have

Fig. 2.26 Integration method (a) and superposition methods (b)–(e) for solving the problem of a statically indeterminate beam



$$EI \frac{d^2 y}{dx^2} = -M_A - R_A x + P(x - a)$$

The boundary conditions are

For $x = 0$, $dy/dx = 0$ and $y = 0$.

For $x = a$, the deflection and the deflection angle are continuous.

For $x = a + b$, $dy/dx = 0$ and $y = 0$.

(2) Superposition method

In the statically indeterminate beam in Fig. 2.26b, the deflection angles at supports A and B are given by

$$\theta_{A1} = \frac{Pb(l^2 - b^2)}{6EI}, \quad \theta_{B1} = \frac{Pa(l^2 - a^2)}{6EI}$$

In the statically indeterminate beam in Fig. 2.26c, the deflection angles at supports A and B are given by

$$\theta_{A2} = \frac{l(2M_A + M_B)}{6EI}, \quad \theta_{B2} = \frac{l(M_A + 2M_B)}{6EI}$$

Then, from $\theta_{A1} + \theta_{A2} = 0$ and $\theta_{B1} + \theta_{B2} = 0$, M_A and M_B are obtained as

$$M_A = -\frac{Pab^2}{l^2}, \quad M_B = -\frac{Pa^2b}{l^2}$$

From the equilibrium of moment at point A, we have

$$M_A - M_B + Pa - R_B l = 0$$

From this, R_A and R_B are obtained as

$$R_A = \frac{Pb^2(l+2a)}{l^3}, \quad R_B = \frac{Pa^2(l+2b)}{l^3}$$

2.4 Strain Energy

We assume that a load P is applied to a beam with a uniform cross-sectional area, and elongation λ is generated. Then, if there is a linear relationship between the load and the elongation, the strain energy U is given as follows (see Fig. 2.27):

$$dU = Pd\lambda, \quad \frac{dU}{d\lambda} = P, \quad U = \int_0^\lambda Pd\lambda = \frac{1}{2}P\lambda \quad (2.38)$$

The strain energy of an elastic bar with a uniform cross-sectional area A and a length l is given by

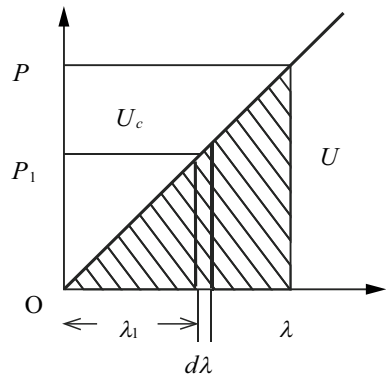
$$U = \frac{1}{2}P\lambda = \frac{P^2 l}{2AE} = \frac{AE}{2l} \lambda^2 \quad (2.39)$$

If there is a linear relationship between the load and the elongation, the tensile strain energy per unit volume U_0 is given by

$$U_0 = (1/2)\sigma\varepsilon = \sigma^2/(2E) = E\varepsilon^2/2 \quad (2.40)$$

The shear strain energy per unit volume U_0' is given by

Fig. 2.27 Strain energy



$$U'_0 = (1/2)\tau\gamma \quad (2.41)$$

By changing the variable from λ to load, we have

$$dU_c = \lambda dP, \quad \frac{dU_c}{dP} = \lambda, \quad U_c = \int_0^P \lambda dP = \frac{1}{2} \lambda P \quad (2.42)$$

The complementary energy U_c can be defined. If there is a linear relationship between the load and the elongation, we have

$$U = U_c$$

2.5 Castigliano's Theorem

In a system where external forces, $P_1, P_2, \dots, P_n$, are applied to an object, the displacement u_i in the direction of the load P_i at the point of application of the load P_i is given by the following equation in terms of the elastic strain energy U and the complementary energy U_c .

$$u_i = \frac{\partial U_c}{\partial P_i}, \quad P_i = \frac{\partial U}{\partial u_i} \quad (2.43)$$

In the case in Fig. 2.27 where there is a linear relationship between the load and the elongation, $U = U_c$. Then, we can rewrite this equation as

$$P_i = \frac{\partial U}{\partial u_i}, \quad u_i = \frac{\partial U}{\partial P_i} \quad (2.44)$$

For example, differentiating by the load P the strain energy of the above-mentioned bar having a uniform cross-sectional area under simple tension, we have

$$\frac{\partial}{\partial P}(U) = \frac{\partial}{\partial P} \left(\frac{P^2 l}{2AE} \right) = \frac{Pl}{AE} = u \quad (2.45)$$

This theorem can be applied to generalized forces, including bending and torsional moments, as the external force P_i . It can also be applied to an external force, as well as generalized displacement including deflection and torsion angles, as the displacement u_i . For example, the following equation can hold:

$$\theta_i = \frac{\partial U}{\partial M_i} \quad (2.46)$$

2.6 Euler's Formula (Buckling of a Long Column)

When the axial compressive load on a column with a perfectly straight axis reaches a certain value, the column becomes unstable and bends. This phenomenon is called buckling, and this load is called the buckling load. Euler theoretically calculated the buckling load P_b and the buckling stress σ_b .

$$P_b = k \frac{\pi^2 EI}{l^2} \quad (2.47)$$

$$\sigma_b = k \frac{\pi^2 E}{k_r^2} \quad (2.48)$$

Figure 2.28 shows columns with different end-fixing conditions.

Case (a): $k = 1/4$.

Case (b): $k = 1$.

Case (c): $k = 4$.

Case (d): $k = 2.046$.

k is called the radius of gyration and k_r is called the slenderness ratio. They are given by the following equations in terms of the length l , cross-sectional area A , moment of inertia I of the column:

$$k = \sqrt{\frac{I}{A}}, \quad k_r = \frac{l}{k} \quad (2.49)$$

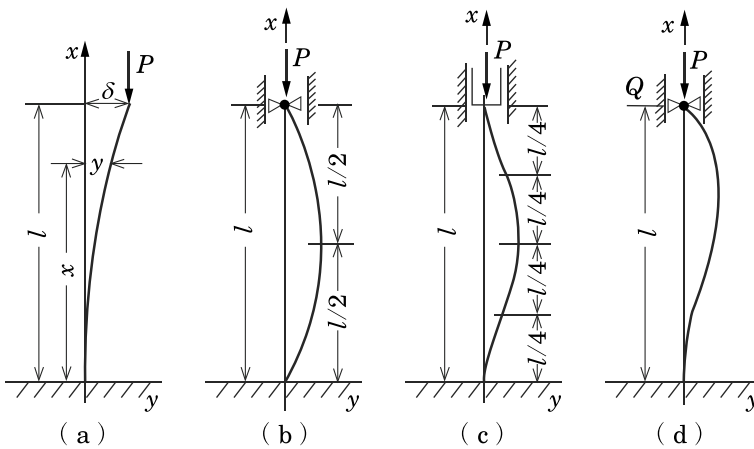


Fig. 2.28 Buckling of a long column

In case (a) in Fig. 2.28, the bending moment M and the axial force N at a distance x are given by the following equations in terms of the deflection of the end of the column δ :

$$M = -P(\delta - y), \quad N = -P$$

The deflection curve is given by

$$\frac{d^2y}{dx^2} = -\frac{M}{EI} = \frac{P}{EI}(\delta - y) \quad (2.50)$$

The general solution of this equation is given by

$$y = C_1 \sin \alpha x + C_2 \cos \alpha x + \delta, \quad \alpha = \sqrt{P/EI}$$

For $x = 0$, $y = 0$ and $dy/dx = 0$. Then, the above equation is rewritten as

$$y = \delta(1 - \cos \alpha x)$$

From $y = \delta$ for $x = l$, we have

$$\delta \cos \alpha l = 0$$

When buckling occurs, $\delta \neq 0$. Then, we have

$$\begin{aligned} \cos \alpha l &= 0 \\ \therefore \alpha l &= (2n+1)\pi/2 \quad (n=0, 1, 2, \dots) \\ \therefore P &= (2n+1)^2 \frac{\pi^2 EI}{4l^2} \end{aligned} \quad (2.51)$$

Since what concerns us is the minimum buckling load, we obtain the following equations for $n = 0$:

$$P_b = \frac{\pi^2 EI}{4l^2} \quad (2.52)$$

$$\sigma_b = \frac{\pi^2 E}{4k^2} \quad (2.53)$$

We consider a straight slender column subjected only to a compressive load. When the load exceeds a certain value, the column suddenly bends laterally. This is the Euler buckling phenomenon. This phenomenon is observed in real structures. For example, suppose you sit on an oil tin. Then, a compressive load is applied to it. If you weigh over a certain amount, the side of the tin bulges outward.

In order to analyze unstable phenomena such as buckling, it is necessary to evaluate the equilibrium of the structure, as well as its stability in the equilibrium state. Figure 2.29 shows the types of problems of elastic stability. (a) is a buckling phenomenon, and point A shows the bifurcation of the equilibrium state. (b) is called snap-through where the state suddenly changes from point A to point B or from point C to point D. An example of this is the case where a spherical shell is subjected to external pressure. (c) is called flattening instability where after the load reaches a certain value (A in the figure), it decreases with increasing deformation. An example of this is when the cross section of a thin round pipe subjected to a bending load is elliptically deformed and its flexural rigidity decreases.

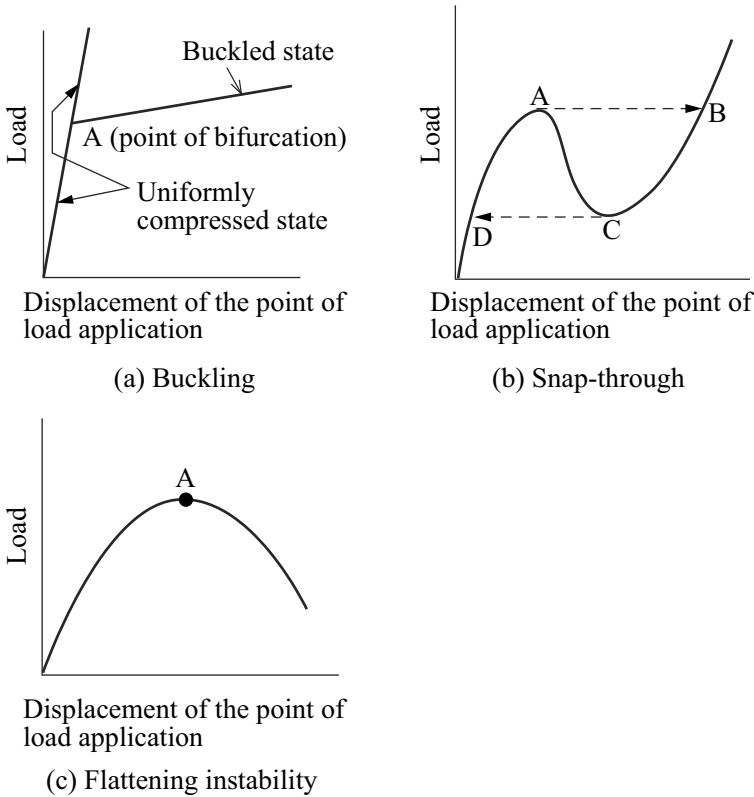
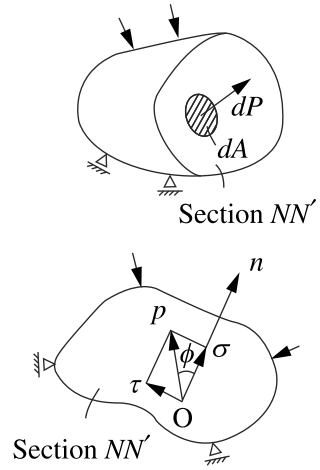


Fig. 2.29 Types of problems of elastic stability [5]

Fig. 2.30 A combined stress

2.7 Combined Stress

The stress generated when two or more of tension, compression, bending, and torsion are applied simultaneously is called a combined stress. Most stresses are combined stresses.

2.7.1 Transformation of Stress

Let p be the stress at a point O , and ϕ be the angle between the normal line and p at point O in the section NN (Fig. 2.30). Then, the normal stress σ is given by

$$\sigma = p \cos \phi \quad (2.54)$$

The shear stress τ is given by

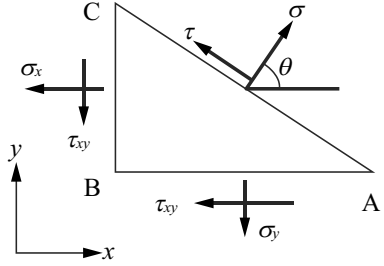
$$\tau = p \sin \phi \quad (2.55)$$

Section NN'

2.7.2 Biaxial Stress State

When, as shown in Fig. 2.31, stresses σ_x , σ_y , and τ_{xy} are generated in the plane normal to the xy -axis in a two-axis Cartesian coordinate system, the stresses σ and τ in the oblique section with the normal line forming an angle θ with the x -axis are given by

Fig. 2.31 Conversion of stress



Let $AC = ds$, $BC = dy$, and $BA = dx$, and assume a unit thickness. θ is the angle between the x -axis and the normal line of AC . Then, we have

$$\frac{dy}{ds} = \cos \theta, \quad \frac{dx}{ds} = \sin \theta \quad (2.56)$$

From the stress equilibrium in the x -direction, we have

$$\sigma \cos \theta \cdot ds - \tau \sin \theta \cdot ds - \sigma_x dy - \tau_{xy} dx = 0 \quad (2.57)$$

From the stress equilibrium in the y -direction, we have

$$\sigma \sin \theta \cdot ds + \tau \cos \theta \cdot ds - \tau_{xy} dy - \sigma_y dx = 0 \quad (2.58)$$

Dividing Eqs. (2.57) and (2.58) by ds and substituting Eq. (2.56), we have

$$\begin{cases} \sigma \cos \theta - \tau \sin \theta - \sigma_x \cos \theta - \tau_{xy} \sin \theta = 0 \\ \sigma \sin \theta + \tau \cos \theta - \tau_{xy} \cos \theta - \sigma_y \sin \theta = 0 \end{cases}$$

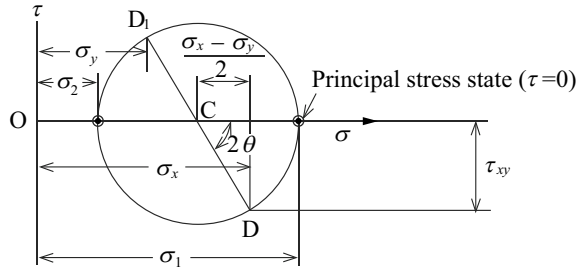
Solving these equations for σ and τ , we have

$$\begin{cases} \sigma = \sigma_x \cos^2 \theta + \sigma_y \sin^2 \theta + 2\tau_{xy} \sin \theta \cos \theta \\ \tau = (\sigma_y - \sigma_x) \sin \theta \cos \theta + \tau_{xy} (\cos^2 \theta - \sin^2 \theta) \end{cases}$$

Rewriting these equations, we have

$$\sigma = \frac{\sigma_x + \sigma_y}{2} + \frac{\sigma_x - \sigma_y}{2} \cos 2\theta + \tau_{xy} \sin 2\theta \quad (2.59)$$

$$\tau = \frac{-\sigma_x - \sigma_y}{2} \sin 2\theta + \tau_{xy} \cos 2\theta \quad (2.60)$$

Fig. 2.32 Mohr's circle of stress

2.7.3 Mohr's Circle of Stress

A stress state in a two-axis Cartesian coordinate system is represented by Mohr's circle of stress described by Fig. 2.32 and the following equation:

$$\left(\sigma - \frac{\sigma_x + \sigma_y}{2} \right)^2 + \tau^2 = \left(\frac{\sigma_x - \sigma_y}{2} \right)^2 + \tau_{xy}^2 \quad (2.61)$$

When the shear stress τ is 0, the normal stresses perpendicular to each other are the maximum and minimum stresses. They are called principal stresses and represented by σ_1 and σ_2 .

$$\sigma_1 = \frac{\sigma_x + \sigma_y}{2} + \sqrt{\left(\frac{\sigma_x - \sigma_y}{2} \right)^2 + \tau_{xy}^2} \quad (2.62)$$

$$\sigma_2 = \frac{\sigma_x + \sigma_y}{2} - \sqrt{\left(\frac{\sigma_x - \sigma_y}{2} \right)^2 + \tau_{xy}^2} \quad (2.63)$$

The plane where principal stresses are applied is called the principal stress plane. From $\tau = 0$, the angle ϕ with the x -axis is given by

$$\tau = \frac{\sigma_x - \sigma_y}{2} \sin 2\phi + \tau_{xy} \cos 2\phi = 0$$

$$\therefore \tan 2\phi = \frac{2\tau_{xy}}{\sigma_x - \sigma_y}$$

2.7.4 Triaxial Stress State

The stress-strain relationship in the three-axis Cartesian coordinate system shown in Fig. 2.13 is given by

$$\left. \begin{aligned}
 \varepsilon_x &= \frac{1}{E} \{ \sigma_x - \nu (\sigma_y + \sigma_z) \} \\
 \varepsilon_y &= \frac{1}{E} \{ \sigma_y - \nu (\sigma_z + \sigma_x) \} \\
 \varepsilon_z &= \frac{1}{E} \{ \sigma_z - \nu (\sigma_x + \sigma_y) \}
 \end{aligned} \right\} \quad (2.64)$$

$$\gamma_{xy} = \frac{1}{G} \tau_{xy}, \quad \gamma_{yz} = \frac{1}{G} \tau_{yz}, \quad \gamma_{zx} = \frac{1}{G} \tau_{zx}$$

2.7.5 Plane Stress and Plane Strain

The plane stress state is equivalent to the state in which, for example, a load is applied to a very thin plate or the free surface of an object. In this state, the stress in the direction of the plate thickness (z -direction) can be ignored, and therefore, $\sigma_z = \tau_{yz} = \tau_{zx} = 0$. Then, the above equations are rewritten as

$$\left. \begin{aligned}
 \varepsilon_x &= \frac{1}{E} (\sigma_x - \nu \sigma_y) & \sigma_x &= \frac{E}{1-\nu^2} (\varepsilon_x + \nu \varepsilon_y) \\
 \varepsilon_y &= \frac{1}{E} (\sigma_y - \nu \sigma_x) & \sigma_y &= \frac{E}{1-\nu^2} (\varepsilon_y + \nu \varepsilon_x) \\
 \varepsilon_z &= -\frac{\nu}{E} (\sigma_x + \sigma_y) & \sigma_z &= 0 \\
 \gamma_{xy} &= \frac{1}{G} \tau_{xy} & \tau_{xy} &= G \gamma_{xy}
 \end{aligned} \right\} \quad (2.65)$$

The plane strain state is equivalent to the internal stress state in a thick material subjected to a load. The strain in the z -direction can be ignored, and therefore, $\varepsilon_z = \gamma_{yz} = \gamma_{zx} = 0$. The stress σ_z in the z -direction exists, and $\sigma_z = \nu (\sigma_x + \sigma_y)$. Inside the material, the strain in the direction normal to the section must be zero to meet the compatibility conditions. Then, we have

$$\left. \begin{aligned}
 \varepsilon_x &= \frac{1}{E} \{ \sigma_x - \nu (\sigma_y + \sigma_z) \} = \frac{1-\nu^2}{E} \left(\sigma_x - \frac{\nu}{1-\nu} \sigma_y \right) \\
 \varepsilon_y &= \frac{1}{E} \{ \sigma_y - \nu (\sigma_z + \sigma_x) \} = \frac{1-\nu^2}{E} \left(\sigma_y - \frac{\nu}{1-\nu} \sigma_x \right) \\
 \gamma_{xy} &= \frac{1}{G} \tau_{xy}
 \end{aligned} \right\} \quad (2.66)$$

2.8 Structural Problems

2.8.1 Thin Cylinder Subjected to Internal Pressure

We consider a two-dimensional plane stress problem. Since we deal with a thin plate, σ_r is much smaller than σ_θ and σ_z and therefore can be ignored. σ_θ and σ_z can be considered to be uniformly distributed across the thickness of the plate. We solve the force equilibrium, constitutive, compatibility equations in this order.

[1] Force Equilibrium Equation

We obtain the hoop stress σ_θ from the force equilibrium in the vertical direction in Fig. 2.33.

$$\begin{aligned} 2\sigma_\theta t &= \int_0^\pi p \sin \theta \cdot R d\theta = 2pR \\ \therefore \sigma_\theta &= \frac{Rp}{t} \end{aligned} \quad (2.67)$$

We obtain the axial stress σ_z from the force equilibrium in the axial direction.

$$\begin{aligned} \pi R^2 p &= 2\pi R t \sigma_z \\ \therefore \sigma_z &= \frac{Rp}{2t} = \frac{\sigma_\theta}{2} \end{aligned} \quad (2.68)$$

[2] The Constitutive Equation.

Let ε_θ be the hoop strain. The following equation is obtained by replacing x, y , and z by r, θ , and z in the strain–stress relationship in the plane stress state in Sect. 2.7.5.

$$\varepsilon_\theta = \frac{1}{E}(\sigma_\theta - \nu \sigma_z) = \frac{\sigma_\theta}{E} \left(1 - \frac{\nu}{2}\right) = \frac{(2-\nu)Rp}{2Et} \quad (2.69)$$

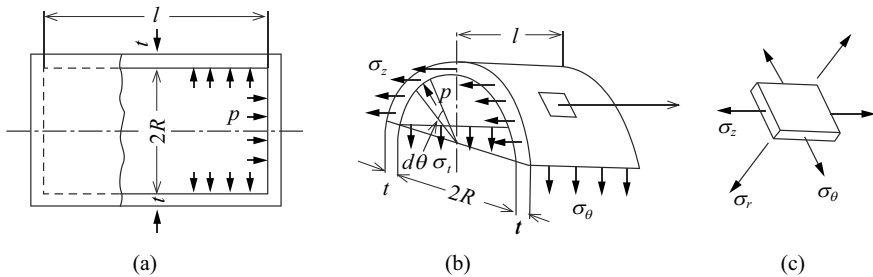


Fig. 2.33 A thin cylinder subjected to internal pressure

[3] The Compatibility Equation

The radial elongation ΔR is given by

$$\Delta R = \varepsilon_{\theta} R = \frac{(2-\nu)R^2 p}{2Et}$$

As with ε_{θ} , the axial strain ε_z is given by

$$\varepsilon_z = \frac{1}{E}(\sigma_z - \nu\sigma_{\theta}) = \frac{\sigma_{\theta}}{E}\left(\frac{1}{2} - \nu\right) = \frac{(1-2\nu)Rp}{2Et} \quad (2.70)$$

The elongation Δl of the overall length l of the cylinder is given by

$$\Delta l = \varepsilon_z l = \frac{(1-2\nu)R/p}{2Et}$$

For a thin spherical shell, the hoop stress σ_{θ} is given by

$$\sigma_{\theta} = \frac{Rp}{2t} \quad (2.71)$$

2.8.2 A Thick Cylinder Subjected to Internal Pressure

Two-dimensional theory of elasticity: We consider a cylinder with a thickness ($r_2 - r_1$) subjected to internal pressure p_1 and external pressure p_2 on the inner and outer surfaces, respectively.

[1] Radial Force Equilibrium

In Fig. 2.34b, radial stress acts on the surfaces ab and cd, and hoop stress acts on the surfaces bc and da.

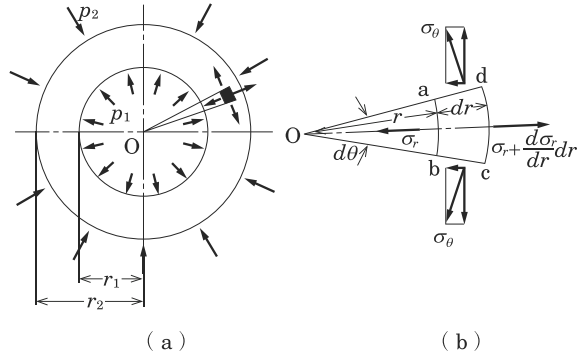
We consider the force equilibrium in abcd. Since the shape and loading conditions are symmetrical in the circumferential direction, we consider only the radial direction.

$$\sigma_r r d\theta - \left(\sigma_r + \frac{d\sigma_r}{dr} dr\right)(r+dr)d\theta + 2\sigma_{\theta} dr \sin \frac{d\theta}{2} = 0 \quad (2.72)$$

For an infinitesimal angle $d\theta$, the third term in the equation can be written as $\sigma_{\theta} dr d\theta$. Substituting this into the above equation to eliminate the term of high order, we obtain

$$\frac{d\sigma_r}{dr} + \frac{\sigma_r - \sigma_{\theta}}{r} = 0 \quad (2.73)$$

Fig. 2.34 A thick cylinder subjected to internal pressure



[2] Strain–Displacement Relationship

The circumference of the radius r expands and shrinks uniformly. Therefore, the displacement at the radius $r + dr$ is u (radial displacement) + $(du/dr)dr$. Then, the radial strain ϵ_r at the radius r is given by

$$\epsilon_r = \frac{u + \frac{du}{dr}dr - u}{dr} = \frac{du}{dr}$$

Since the circumference of the radius r changes from $2\pi r$ to $2\pi(r + u)$, the hoop strain ϵ_θ is given by

$$\epsilon_\theta = \frac{2\pi(r + u) - 2\pi r}{2\pi r} = \frac{u}{r}$$

[3] Stress–Strain Relationship

$$\sigma_r = \frac{E}{1-\nu^2}(\epsilon_r + \nu\epsilon_\theta) = \frac{E}{1-\nu^2}\left(\frac{du}{dr} + \nu\frac{u}{r}\right)$$

$$\sigma_\theta = \frac{E}{1-\nu^2}(\epsilon_\theta + \nu\epsilon_r) = \frac{E}{1-\nu^2}\left(\frac{u}{r} + \nu\frac{du}{dr}\right)$$

Substituting these into the force equilibrium Eq. (2.70) and rearranging the equation, we obtain

$$\frac{d^2u}{dr^2} + \frac{1}{r} \cdot \frac{du}{dr} - \frac{u}{r^2} = 0$$

By rewriting this equation, we obtain

$$\frac{d^2u}{dr^2} + \frac{d}{dr}\left(\frac{u}{r}\right) = 0$$

Integrating this equation, we obtain

$$\frac{du}{dr} + \frac{u}{r} = C \quad \text{or} \quad r \frac{du}{dr} + u = Cr \quad \text{therefore} \quad \frac{d}{dr}(ur) = Cr$$

Further integrating it, we obtain

$$u = c_1 \cdot r + c_2/r$$

From this, we obtain the general solution for stress:

$$\sigma_r = \frac{E}{1-\nu^2} \left\{ (1+\nu)c_1 - (1-\nu)c_2 \frac{1}{r^2} \right\}$$

$$\sigma_\theta = \frac{E}{1-\nu^2} \left\{ (1+\nu)c_1 + (1-\nu)c_2 \frac{1}{r^2} \right\}$$

[4] Boundary Conditions

Since $\sigma_r = -p_1$ for $r = r_1$, and $\sigma_r = -p_2$ for $r = r_2$, we have

$$c_1 = \frac{1-\nu}{E} \cdot \frac{r_1^2 p_1 - r_2^2 p_2}{r_2^2 - r_1^2} = \frac{1-\nu}{E} \cdot \frac{n^2 p_1 - p_2}{1-n^2}$$

$$c_2 = \frac{1+\nu}{E} \cdot \frac{r_1^2 r_2^2 (p_1 - p_2)}{r_2^2 - r_1^2} = \frac{1+\nu}{E} \cdot \frac{r_1^2 (p_1 - p_2)}{1-n^2}$$

Here, $n = r_1/r_2$.

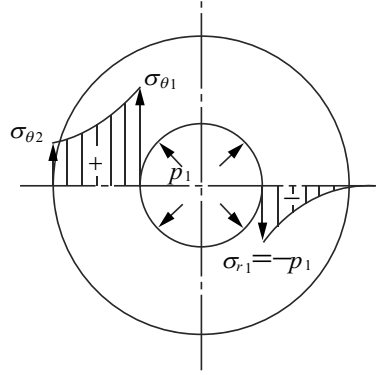
Substituting these into the equation for the general solution, we obtain

$$\left. \begin{aligned} \sigma_r &= \frac{n^2 p_1 - p_2}{1-n^2} - \frac{p_1 - p_2}{1-n^2} \cdot \frac{r_1^2}{r^2} \\ \sigma_\theta &= \frac{n^2 p_1 - p_2}{1-n^2} + \frac{p_1 - p_2}{1-n^2} \cdot \frac{r_1^2}{r^2} \end{aligned} \right\} \quad (2.74)$$

The radial displacement u at the radius r is given by

$$u = \frac{1-\nu}{E} \cdot \frac{n^2 p_1 - p_2}{1-n^2} r + \frac{1+\nu}{E} \cdot \frac{p_1 - p_2}{1-n^2} \frac{r_1^2}{r}$$

Fig. 2.35 A cylinder subjected only to internal pressure



[5] Cylinder Subjected only to Internal Pressure ($p_2 = 0$)

$$\left. \begin{aligned} \sigma_r &= \frac{p_1}{1-n^2} \left(\frac{r_1^2}{r^2} - n^2 \right) \\ \sigma_\theta &= \frac{p_1}{1-n^2} \left(\frac{r_1^2}{r^2} + n^2 \right) \end{aligned} \right\} \quad (2.75)$$

Since $r_1/r > n$, $\sigma_r < 0$ and $\sigma_\theta > 0$.

As shown in Fig. 2.35, σ_θ becomes maximum at the inner surface ($r = r_1$) and the maximum stress $\sigma_{\theta \max}$ is given by

$$\sigma_{\theta \max} = \frac{1+n^2}{1-n^2} p_1$$

The maximum shear stress $\tau_{\max}$ occurs at the surface and is given by

$$\tau_{\max} = \frac{\sigma_{\theta \max} - \sigma_{r1}}{2} = \frac{p_1}{1-n^2}$$

Substituting $p_2 = 0$ and $r = r_1$, the displacement of the inner surface is given by

$$u_1 = \frac{(1-\nu)r_1^2 + (1+\nu)r_2^2}{E(r_2^2 - r_1^2)} r_1 p_1 = \frac{p_1 r_1}{E} \left(\frac{r_2^2 + r_1^2}{r_2^2 - r_1^2} + \nu \right)$$

[6] A BWR Reactor Pressure Vessel

We consider a BWR reactor pressure vessel with an inner diameter of $r_1 = 3,200$ mm, a wall thickness of 160 mm, and an internal pressure of $p_1 = 7$ MPa. The hoop stress σ_θ (at the inner surface) and σ_θ (at the outer surface) are given by ($n = 3200/3360 = 1/1.05$)

$$\sigma_\theta \text{ (inner surface)} = \frac{p_1}{1-n^2} \left(\frac{r_1^2}{r^2} + n^2 \right) = 143.6 \text{ MPa}$$

$$\sigma_\theta \text{ (outer surface)} = \frac{p_1}{1-n^2} \left(\frac{r_1^2}{r^2} - n^2 \right) = 136.6 \text{ MPa}$$

Treating the vessel as a thin cylinder and substituting $R = 3200$ in $\sigma_\theta = Rp/t$, we obtain.

$$\sigma_\theta \text{ (thin wall)} = 140.0 \text{ MPa} \dots \text{<Primary general membrane stress>}$$

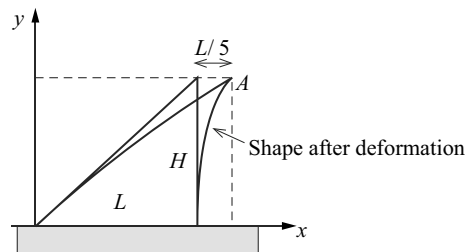
Substituting $R = 3280$ (average radius), we obtain

$$\sigma_\theta \text{ (thin wall)} = 143.5 \text{ MPa.}$$

2.8.3 Plane Stress, Strain Energy, and Castigliano's Theorem

[Example Problem] This problem concerns the theoretical basics of the finite element method based on the title of this chapter. We consider a right-angled triangular thin plate, with a thickness t , a bottom length L , and a height H , constrained at the bottom as shown in Fig. 2.36. A load (P_x, P_y) is applied at the vertex to deform it by displacement (u_x^A, u_y^A). We assume infinitesimal linear elastic deformation and a linearly distributed displacement field. We also assume the following stress–strain relationship in the plane stress state.

Fig. 2.36 Deformation of a thin triangular plate



Shape after deformation.

$$\begin{pmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{pmatrix} = \frac{E}{1-\nu^2} \begin{bmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & \frac{1-\nu}{2} \end{bmatrix} \begin{pmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{pmatrix} \quad (2.76)$$

where E is Young's modulus and ν is Poisson's ratio.

- (1) Express the displacement (u_x, u_y) in the plate in terms of (u_x^A, u_y^A) and the coordinate (x, y) .
- (2) Obtain the strain $(\varepsilon_x, \varepsilon_y, \gamma_{xy})$ and the stress $(\sigma_x, \sigma_y, \tau_{xy})$ in the plate.
- (3) Obtain the strain energy U in the plate.
- (4) Express the load (P_x, P_y) on the vertex A in terms of the displacement (u_x^A, u_y^A) of the point of load application.
- (5) Consider the displacement $(u_x^A = L/5, u_y^A = 0)$ of point A without the assumption of a linearly distributed displacement field. Sketch the deformed shape to describe how the plate is deformed, and explain why you think it is deformed like this.

[Hint] When linear elasticity and plane stress are assumed, there are the following relationships among the strain energy U , the stress field $(\sigma_x, \sigma_y, \tau_{xy})$, the strain field $(\varepsilon_x, \varepsilon_y, \gamma_{xy})$, the displacement field (u_x, u_y) , and the thickness t in the given object region.

$$U = \frac{t}{2} \iint_{\text{領域}} (\sigma_x \varepsilon_x + \sigma_y \varepsilon_y + \tau_{xy} \gamma_{xy}) dx dy \quad (2.77)$$

$$\varepsilon_x = \frac{\partial u_x}{\partial x}, \quad \varepsilon_y = \frac{\partial u_y}{\partial y}, \quad \gamma_{xy} = \frac{\partial u_x}{\partial y} + \frac{\partial u_y}{\partial x} \quad (2.78)$$

$$\left. \begin{aligned} \sigma_x &= \frac{E}{1-\nu^2} (\varepsilon_x + \nu \varepsilon_y), \quad \sigma_y = \frac{E}{1-\nu^2} (\varepsilon_y + \nu \varepsilon_x) \\ \tau_{xy} &= \frac{E}{2(1+\nu)} \gamma_{xy} \end{aligned} \right\} \quad (2.79)$$

From Castigliano's theorem, we have the relationship $P = \partial U / \partial u$ between the load P on a given point and the displacement u in the direction of P :

[Example Answer] (1) Assuming linear elastic deformation, we have

$$\left. \begin{aligned} u_x &= a_1x + b_1y + c_1 \\ u_y &= a_2x + b_2y + c_2 \end{aligned} \right\} \quad (2.80)$$

where a_1, a_2, b_1, b_2, c_1 , and c_2 are constants. Since the plate is fixed at the bottom, along the line $(x, 0)$, we have

$$u_x(x, 0) = u_y(x, 0) = 0$$

From Eq. (2.80), we have.

$a_1x + c_1 = 0$. Thus, $a_1 = 0$ and $c_1 = 0$.

$a_2x + c_2 = 0$. Thus, $a_2 = 0$ and $c_2 = 0$.

At point (L, H) ,

$$u_x(L, H) = u_x^A, \quad u_y(L, H) = u_y^A$$

Then, from Eq. (2.28), we have

$$\begin{aligned} u_x(L, H) &= b_1H = u_x^A \\ u_y(L, H) &= b_2H = u_y^A \end{aligned}$$

Then, we have

$$b_1 = \frac{u_x^A}{H}, \quad b_2 = \frac{u_y^A}{H}$$

Hence, we obtain

$$u_x = \frac{u_x^A}{H} y, \quad u_y = \frac{u_y^A}{H} y$$

- (2) From the answer in (1) and the stress–strain relationship in the plane stress state, we have

$$\begin{aligned}
 \varepsilon_x &= \frac{\partial u_x}{\partial x} = 0 \\
 \varepsilon_y &= \frac{\partial u_y}{\partial y} = \frac{u_y^A}{H} \\
 \gamma_{xy} &= \frac{\partial u_x}{\partial y} + \frac{\partial u_y}{\partial x} = \frac{u_x^A}{H} \\
 \sigma_x &= \frac{E}{1-\nu^2}(\varepsilon_x + \nu\varepsilon_y) = \frac{E\nu}{1-\nu^2} \cdot \frac{u_y^A}{H} \\
 \sigma_y &= \frac{E}{1-\nu^2}(\varepsilon_y + \nu\varepsilon_x) = \frac{E}{1-\nu^2} \cdot \frac{u_y^A}{H} \\
 \tau_{xy} &= \frac{E}{2(1+\nu)}\gamma_{xy} = \frac{Eu_x^A}{2(1+\nu)H}
 \end{aligned}$$

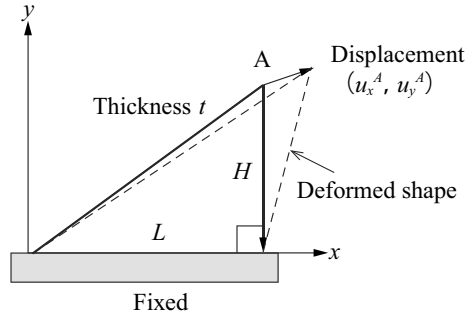
- (3) The strain energy U is given by

$$\begin{aligned}
 U &= \frac{t}{2} \iint (\sigma_x \varepsilon_x + \sigma_y \varepsilon_y + \tau_{xy} \gamma_{xy}) dx dy \\
 &= t \left\{ \frac{E}{1-\nu^2} \cdot \frac{(u_y^A)^2}{H^2} + \frac{E}{2(1+\nu)} \cdot \frac{(u_x^A)^2}{H^2} \right\} \cdot \frac{1}{2} \int_0^L \int_0^H dx dy \\
 &= \frac{1}{2} \left\{ \frac{(u_y^A)^2}{1-\nu^2} + \frac{(u_x^A)^2}{2(1+\nu)} \right\} \cdot \frac{E}{H^2} \cdot \frac{L \cdot H}{2} \\
 &= \frac{t}{4} \left\{ \frac{(u_y^A)^2}{1-\nu^2} + \frac{(u_x^A)^2}{2(1+\nu)} \right\} \frac{LE}{H}
 \end{aligned}$$

- (4) Using the strain energy U obtained in (3), from Castigliano's theorem, we have (Fig. 2.37)

$$\begin{aligned}
 P_x &= \frac{\partial U}{\partial u_x^A} = \frac{tu_x^A}{4(1+\nu)} \cdot \frac{LE}{H} \\
 P_y &= \frac{\partial U}{\partial u_y^A} = \frac{tu_y^A}{2(1-\nu^2)} \cdot \frac{LE}{H}
 \end{aligned}$$

Fig. 2.37 Deformation by linear approximation



2.9 Principle of Energy [1–3]

The principle of energy provides a theoretical base for mechanics and is essentially equivalent to analytical mechanics. There is the principle of virtual work. In textbooks on analytical mechanics, it is given by

$$-\frac{\partial U_{\text{ex}}}{\partial u} = f_{\text{ex}} \quad (2.81)$$

where U_{ex} is the potential energy, u is the displacement, and f_{ex} is the conservative force (the force without energy dissipation or the force from interaction with the environment). In textbooks on strength of materials and structural mechanics, the principle of virtual work is given by

$$\frac{\partial U_{\text{in}}}{\partial u} = f_{\text{ex}} \quad (2.82)$$

where U_{in} is the strain energy and f_{ex} is the external force.

Below we try to explain what causes the difference in sign between Eqs. (2.81) and (2.82).

Physical analytical mechanics begins with the principle of minimum action. Here, we describe the movement of a small non-deforming particle that is treated as a point mass or a rigid body. Let $T = (1/2) mu^2$ be the kinetic energy of the particle (m : mass, u : displacement, and $\dot{\bullet}$: time derivative) and U_{ex} be the potential energy of interaction with the environment. Then, the Lagrangian function L and the action S are defined by

$$L = T - U_{\text{ex}}$$

$$S = \int L dt$$

The principle of minimum action is that the movement of the particle is determined to minimize the action S . In different movement patterns $(u + \delta u, u + \delta u^*)$, the

movement is determined to minimize S or make the variation δS zero. That means, $\delta S = 0$ for the minimum S , but S can be maximized for $\delta S = 0$. In other words, $\delta S = 0$ is only a requirement for minimizing S . Then, we obtain the following Lagrangian equation:

$$\frac{d}{dt} \left(\frac{\partial L}{\partial \dot{u}} \right) - \frac{\partial L}{\partial u} = 0 \quad (2.83)$$

Substituting the kinetic energy equation into Eq. (2.83), we obtain the equation of motion

$$m\ddot{u} = -\frac{\partial U_{\text{ex}}}{\partial u} \quad (2.84)$$

In the above, we consider that when subjected to an external force, the particle moves without being deformed. Strength of materials and structural mechanics deal with problems in which an object deforms or vibrates while its center remains nearly unchanged due to mechanical constraints. This is described in Fig. 2.31.

In strength of materials and structural mechanics, the deformation of an object is considered, and therefore the potential energy must be considered to consist of U_{ex} by the interaction with the environment, and the strain energy U_{in} .

$$U = U_{\text{ex}} + U_{\text{in}} \quad (2.85)$$

In the material in Fig. 2.38b, the external force f_{ex} and the internal force f_{in} are in equilibrium, that is,

$$f_{\text{in}} + f_{\text{ex}} = 0 \quad (2.86)$$

The reaction-strain energy relationship is given by the following equation, which is similar to Eq. (2.81):

$$-\frac{\partial U_{\text{in}}}{\partial u} = f_{\text{in}} \quad (2.87)$$

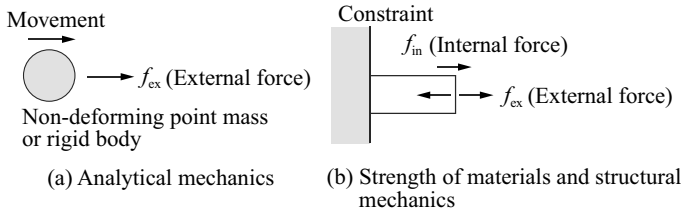


Fig. 2.38 Movement and deformation by external force in analytical mechanics and structural mechanics

Eliminating f_{in} , we obtain

$$\frac{\partial U_{\text{in}}}{\partial u} = f_{\text{ex}} \quad (2.88)$$

Thus, we obtained the principle of virtual work for strength of materials and structural mechanics.

Equation (2.88) can be said to be the solution to minimize the total potential energy Π ($\delta\Pi = 0$) given by

$$\Pi = U_{\text{in}} - f_{\text{ex}}u \quad (2.89)$$

where $-f_{\text{ex}}u$ is the potential energy due to the external force and corresponds to U_{ex} . This principle is called the principle of minimum potential energy. In textbooks on solid, material, and structural mechanics, a volume or surface force is used as an external force, and the left-hand side of the equation is described by the sum of the volume integral of the strain energy and the volume or surface integral of the product of the external force and the displacement. The strain energy U_{in} can be given by the integral of load over displacement.

$$U_{\text{in}} = \int f_{\text{ex}} du \quad (2.90)$$

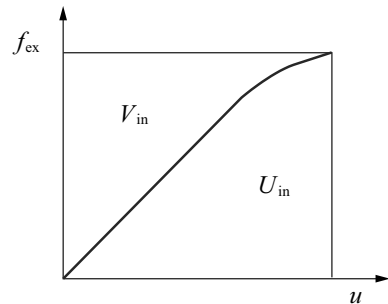
In Eq. (2.90), displacement is a variable, and therefore, this principle is also called the displacement method. U_{in} is the strain energy and corresponds to the bottom half of the area in Fig. 2.39

Also, there is a principle in which the total complementary energy Π' represented by the top half of the area V_{in} in Fig. 2.38 is defined as

$$\Pi' = V_{\text{in}} - f_{\text{ex}}u \quad (2.91)$$

and the force is a variable defined as

Fig. 2.39 Strain energy U_{in} and complementary energy V_{in}



$$\frac{\partial V_{in}}{\partial f_{ex}} = u \quad (2.92)$$

This is called the principle of minimum complementary energy, the principle of complementary virtual work, or the stress method.

Additionally, we discuss the physical significance of action as a functional in analytical mechanics. In a system without energy loss, action can be transformed to

$$\begin{aligned} S &= \int (T - U_{ex}) dt \\ &= \int (2T - E) dt \end{aligned} \quad (2.93)$$

where E is the energy ($= T + U_{ex}$).

Using the de Broglie relation,

$$p = \hbar k, \quad E = \hbar \omega$$

(p : momentum, $\hbar$: Planck's constant/ 2π , k : wave number, ω : angular frequency).

Equation (2.93) can be written as

$$S = px - Et = \hbar(kx - \omega t) \quad (2.94)$$

The term in parentheses in Eq. (2.94) describes the phase of the wave $\theta = kx - \omega t$. That waves travel in a way to minimize the phase is a basic principle of wave mechanics, geometrical optics, and quantum mechanics. An action is like a phase. Particles move in a way to minimize the action.

Now, you understand that seemingly different academic areas are based on the principle of energy. That principles in different areas correspond to each other is called the correspondence principle.

Appendix: General and Engineering Notations and Tensor and Vector Notations of Stress and Strain [7]

For simplicity, we consider a two-dimensional system.

The direction cosine matrix between coordinate axes is expressed using the following symbols:

$$\begin{bmatrix} x' \\ y' \end{bmatrix} = \begin{bmatrix} l_1 & m_1 \\ l_2 & m_2 \end{bmatrix} \begin{bmatrix} x \\ y \end{bmatrix} \quad (2.95)$$

Then, the stress components in the new coordinate system (x' , y' , z') can be expressed by the following coordinate transformation formula:

$$\begin{bmatrix} \sigma_{x'x'} & \sigma_{y'x'} \\ \sigma_{x'y'} & \sigma_{y'y'} \end{bmatrix} = \begin{bmatrix} l_1 & m_1 \\ l_2 & m_2 \end{bmatrix} \begin{bmatrix} \sigma_{xx} & \sigma_{yx} \\ \sigma_{xy} & \sigma_{yy} \end{bmatrix} \begin{bmatrix} l_1 & l_2 \\ m_1 & m_2 \end{bmatrix} \quad (2.96)$$

As opposed to the coordinate transformation in Equation (2.95), quantities governed by the transformation principle in Equation (2.96) are called tensors (second order) in mathematics.

In other words, stress components showing the state of an internal force at a single point of an elastic body are tensors (second order). These are called stress tensors.

Similarly, the transformation of physical strain components can be expressed by

$$\begin{bmatrix} \varepsilon_{x'x'} & \varepsilon_{y'x'} \\ \varepsilon_{x'y'} & \varepsilon_{y'y'} \end{bmatrix} = \begin{bmatrix} l_1 & m_1 \\ l_2 & m_2 \end{bmatrix} \begin{bmatrix} \varepsilon_{xx} & \varepsilon_{yx} \\ \varepsilon_{xy} & \varepsilon_{yy} \end{bmatrix} \begin{bmatrix} l_1 & l_2 \\ m_1 & m_2 \end{bmatrix} \quad (2.97)$$

Equation (2.97) follows the same transformation principle for Equation (2.96). Therefore, strain components are tensors (second order), and Equation (2.98) is called a strain tensor equation. Then, we have

$$\begin{bmatrix} \varepsilon_{xx} & \varepsilon_{yx} \\ \varepsilon_{xy} & \varepsilon_{yy} \end{bmatrix} \quad (2.98)$$

$$\varepsilon_{xx} = \frac{\partial u_x}{\partial x}, \quad \varepsilon_{yy} = \frac{\partial u_y}{\partial y}, \quad \varepsilon_{xy} = \varepsilon_{yx} = \frac{1}{2} \left(\frac{\partial u_x}{\partial y} + \frac{\partial u_y}{\partial x} \right)$$

The constitutive conditions are

$$\begin{aligned} \varepsilon_x &= \frac{\sigma_x}{E} - \frac{\nu \sigma_y}{E} \\ \varepsilon_y &= \frac{\sigma_y}{E} - \frac{\nu \sigma_x}{E} \\ \sigma_x &= \frac{E}{1-\nu^2} (\varepsilon_x + \nu \varepsilon_y) \\ \sigma_y &= \frac{E}{1-\nu^2} (\varepsilon_y + \nu \varepsilon_x) \\ \sigma_{xy} &= \frac{E}{1+\nu} \varepsilon_{xy} \end{aligned}$$

These can be generalized as

$$\sigma_{ii} = E_{ijkl} \varepsilon_{kl} \quad (2.99)$$

elastic constant tensor (fourth order)

Here, two subscripts mean summation. $\sum$ is omitted in the equations. In a two-dimensional case, these equations can be written as

$$\begin{aligned} E_{1111} &= \frac{E}{1-\nu^2}, \quad E_{1112} = E_{1121} = 0, \quad E_{1122} = \frac{\nu E}{1-\nu^2}, \\ E_{1211} &= E_{1221} = E_{1222} = 0, \quad E_{1212} = \frac{E}{1+\nu}, \quad \text{etc} \end{aligned}$$

Here, the strain energy per unit volume u_e is generally given by

$$u_e = \sum_i \sum_j \sigma_{ij} \varepsilon_{ij} \equiv \sigma_{ij} \varepsilon_{ij} \quad (2.100)$$

In a two-dimensional case, this can be written as

$$u_e = \sigma_{xx} \varepsilon_{xx} + \sigma_{yy} \varepsilon_{yy} + \sigma_{xy} \varepsilon_{xy} + \sigma_{yx} \varepsilon_{yx} \quad (2.101)$$

If the space is uniform and there is no difference in the x - and y -directions, the equation can be written as

$$u_e = \sigma_{xx} \varepsilon_{xx} + \sigma_{yy} \varepsilon_{yy} + 2\sigma_{xy} \varepsilon_{xy}$$

In the engineering notation, it can be written as

$$\begin{bmatrix} \sigma_x & \tau_{xy} \\ \tau_{yx} & \sigma_y \end{bmatrix}, \begin{bmatrix} \varepsilon_x & \gamma_{xy}/2 \\ \gamma_{yx}/2 & \varepsilon_y \end{bmatrix}$$

If the space is uniform, this can be written as

$$\begin{aligned} \gamma_{xy} &= \gamma_{yx} = \frac{\partial v}{\partial x} + \frac{\partial u}{\partial y} \\ \tau_{xy} &= \frac{E}{2(1+\nu)} \gamma_{xy} \end{aligned}$$

Now, we get,

$$\begin{aligned} \varepsilon_{xy} &= \frac{1}{2} \left(\frac{\partial v}{\partial x} + \frac{\partial u}{\partial y} \right) = \frac{1}{2} \gamma_{xy} \\ \sigma_{xy} &= \frac{E}{1+\nu} \varepsilon_{xy} = \frac{E}{2(1+\nu)} \gamma_{xy} = \tau_{xy} \end{aligned}$$

The strain energy per unit volume is given by

$$u_e = \sigma_x \varepsilon_x + \sigma_y \varepsilon_y + \frac{\tau_{xy} \gamma_{xy}}{2} + \frac{\tau_{yx} \gamma_{yx}}{2} \quad (2.102)$$

If the space is uniform, this can be written as

$$u_e = \sigma_x \varepsilon_x + \sigma_y \varepsilon_y + \tau_{xy} \gamma_{xy} \quad (2.103)$$

In other words, in the general and engineering notations, the value of strain changes as follows. For simplicity, in strength of materials in engineering, both stress and strain are treated as vectors, taking into account their symmetry.

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} = \begin{bmatrix} \frac{E}{1-\nu^2} & -\frac{\nu E}{1-\nu^2} & 0 \\ -\frac{\nu E}{1-\nu^2} & \frac{E}{1-\nu^2} & 0 \\ 0 & 0 & \frac{E}{2(1+\nu)} \end{bmatrix} \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} \quad (2.104)$$

This simplification reduces the elastic constant tensor to a second-order tensor, making it easier to express the tensor. The strain energy per unit volume can be given by the inner product of the vectors.

$$u_e = \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} \cdot \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{Bmatrix} = \sigma_x \varepsilon_x + \sigma_y \varepsilon_y + \tau_{xy} \gamma_{xy} \quad (2.105)$$

Exercises for This Chapter

Thin Cylinder, Spherical Shell, and Truss Problems

1. Calculate hoop and axial stresses in a cylindrical pressure vessel with an inner diameter of 3.5 m and a wall thickness of 210 mm, subjected to an internal pressure of 15.0 MPa. The cylinder is closed at both ends. Discontinuity stresses can be ignored. Derive and describe the relationship between the internal pressure and the stresses.
2. An internal pressure of 3.92 MPa is applied to a spherical pressure vessel with an inner diameter of 5 m.

Given an allowable stress of 118 MPa, what is the minimum thickness of the vessel? How much does the volume of the vessel increase? The Young's modulus of the vessel is 206 GPa and Poisson's ratio is 0.3.

Also, derive and describe the relationship between the internal pressure and stress.

3. (1) Consider a vessel with a radius r , a length h and a thickness t , and with semi-spherical ends with a radius r and a thickness t , as shown in Fig. 1. Calculate the hoop membrane stress σ_θ and the axial membrane stress σ_z in the cylindrical shell when an internal pressure P is applied to the vessel. Also, show how you got the answer. Assume that the vessel is a thin-wall structure ($t/r < 0.1$). Also, assume that h is sufficiently long and σ_θ and σ_z are constant.
- (2) Consider that as shown in Fig. 2 or 3, there is one semi-elliptical crack with a length $2c$ and a depth a in the inner surface of the cylinder. Which is more dangerous, a circumferential crack (in the θ -direction) or an axial crack (in the z -direction)? Also, explain the reason.

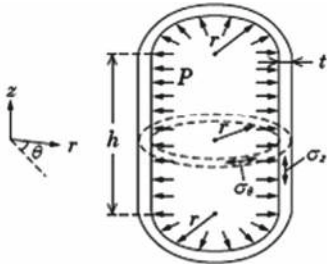


Figure 1 • A cylindrical vessel with semi-spherical ends subjected to an internal pressure

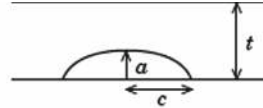


Figure 2 • A semi-elliptical axial surface crack

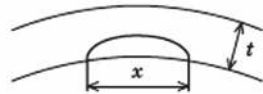
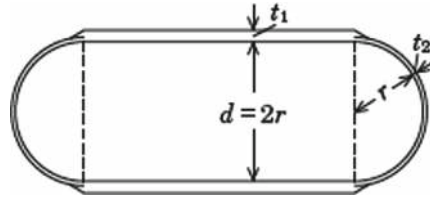


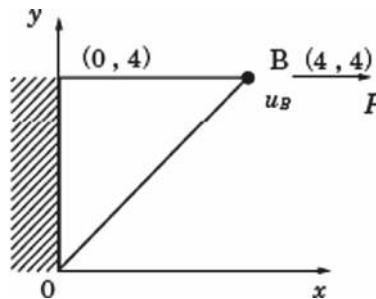
Figure 3 • A semi-elliptical axial surface crack

4. As shown in the figure, an internal pressure P is applied to a thin cylindrical vessel (an inner diameter $d = 2r$ and a thickness t_1) with semi-spherical heads (an inner diameter $d = r$ and a thickness t_2).
- (1) Derive an equation for the increase in the primary membrane stress in the cylindrical shell and the head and in the radius. Let E be the Young's modulus of the material and ν be Poisson's ratio.
- (2) Determine the ratio of t_1/t_2 that makes equal the primary membrane stresses in the cylinder and the head.
- (3) Determine the ratio of t_1/t_2 that prevents secondary stresses in the cylindrical shell and the head. Let $\nu = 0.3$.



Plane Stress and Strain Energy Problems

5. Consider a triangular structure subjected to a concentrated load P at point B as shown in the figure. Let u_B be the deflection of point B and t be the thickness. The deflection of the side on the y -axis is constrained. Answer the following questions:
 - (1) Write an expression using the relationship between the deflection (u, v) and the strain ($\epsilon_x, \epsilon_y, \gamma_{xy}$), the relationship between the strain ($\epsilon_x, \epsilon_y, \gamma_{xy}$) and the stress ($\sigma_x, \sigma_y, \tau_{xy}$), and the stress and strain obtained from strain energy in the plane stress state.
 - (2) Assume that deflection in the structure occurs only in the x -direction and can be approximated as $u = ax + by + c$ and $v = 0$. Calculate the strain energy and describe u_B with P using Castigliano's theorem.
 - (3) Descriptively sketch the shape of the structure after deformation.
6. Consider a rectangle OABC with sides OA and OC fixed, subjected to loads (P_B, Q_B) at point B, as shown in the figure. Let l be the thickness and E be Young's modulus, and ν be Poisson's ratio. Describe the displacement of point B (u_B, v_B) as a function of (P_B, Q_B). Assume linear elasticity and plane stress. Also, assume the displacement field in the rectangle as a simple function if necessary. Obtain the value of u_B for $E = 1$, $\nu = 0.3$, and $P_B = Q_B = 1$.



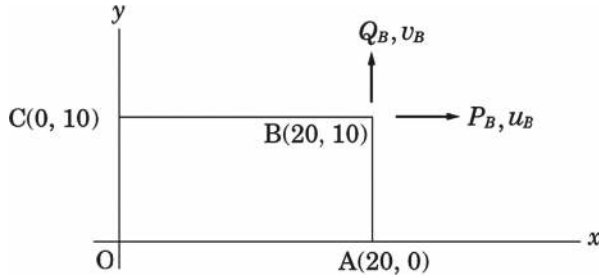
Hint: Let U be the strain energy, $(\sigma_x, \sigma_y, \sigma_{xy})$ be the stress field, $(\epsilon_x, \epsilon_y, \epsilon_{xy})$ be the strain field, (u, v) be the displacement field, and t be the thickness. Then, there are the following relationships in the given object region when linear elasticity and plane stress are assumed.

$$U = \frac{t}{2} \iint_{\text{region}} (\sigma_x \epsilon_x + \sigma_y \epsilon_y + 2\sigma_{xy} \epsilon_{xy}) dx dy$$

$$\epsilon_x = \frac{\partial u}{\partial x}, \quad \epsilon_y = \frac{\partial v}{\partial y}, \quad \epsilon_{xy} = \frac{1}{2} \left(\frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \right)$$

$$\sigma_x = \frac{E}{1-\nu^2} (\epsilon_x + \nu \epsilon_y), \quad \sigma_y = \frac{E}{1-\nu^2} (\epsilon_y + \nu \epsilon_x), \quad \sigma_{xy} = \frac{E}{1+\nu} \epsilon_{xy}$$

Let R be the load on any point and ω be the displacement in the same direction as the load R . Then, there is relationship $R = \partial U / \partial \omega$.



7. (1) Consider a right-angled triangle with the side OA and OB fixed, subjected to a load Q_D in the y -direction at middle point D on the side AB. Let l be the thickness and E be Young's modulus, ν be Poisson's ratio, and (u_D, v_D) be the displacement of point D. Describe the strain energy U in the triangle OAB as a function of (u_D, v_D) . Assume linear elasticity and plane stress. Also, assume a linear displacement distribution in the triangles OAD and OBD.
- (2) Obtain the relationship between v_D and Q_D . Also, obtain the value of v_D for $E = 1$, $\nu = 0.3$, and $Q_D = 1$.

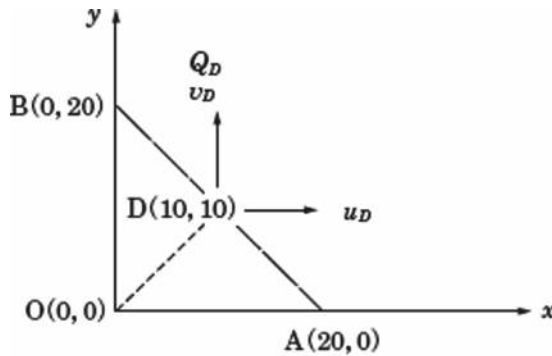


Figure • The system, load, and displacement

Hint: Let U be the strain energy, $(\sigma_x, \sigma_y, \sigma_{xy})$ be the stress field, $(\varepsilon_x, \varepsilon_y, \varepsilon_{xy})$ be the strain field, (u, v) be the displacement field, and t be the thickness. Then, there are the following relationships in the given object region when linear elasticity and plane stress are assumed.

$$U = \frac{t}{2} \iint_{\text{region}} (\sigma_x \varepsilon_x + \sigma_y \varepsilon_y + 2\sigma_{xy} \varepsilon_{xy}) dx dy$$

$$\varepsilon_x = \frac{\partial u}{\partial x}, \quad \varepsilon_y = \frac{\partial v}{\partial y}, \quad \varepsilon_{xy} = \frac{1}{2} \left(\frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \right)$$

$$\sigma_x = \frac{E}{1-\nu^2} (\varepsilon_x + \nu \varepsilon_y), \quad \sigma_y = \frac{E}{1-\nu^2} (\varepsilon_y + \nu \varepsilon_x)$$

$$\sigma_{xy} = \frac{E}{1+\nu} \varepsilon_{xy}$$

Let Q be the load on any point and v be the displacement in the direction of the load Q . Then, there is relationship $Q = \partial U / \partial v$.

Thermal Stress Problem

8. Consider the problem of thermal deformation of two different metal plates with different coefficients of linear expansion and a length L , subjected to a temperature increase ΔT , as shown in Figs. 1 and 2. Answer the following questions. Assume that the metal plates are not stuck to each other and can move freely against each other. Do not consider deformation in the direction of the thickness and width. Let α_1 , E_1 , and A_1 be the coefficient of linear expansion, Young's modulus, and cross-sectional area of metal plate 1, respectively, and α_2 , E_2 , and A_2 be the coefficient of linear expansion, Young's modulus, and cross-sectional area of metal plate 2, respectively.
 - (1) Assume that as shown in Fig. 1, the plates are fixed at end A and free at end B. How much do the metal plates expand in the direction of the length due to the increase in temperature T ?
 - (2) Assume that as shown in Fig. 2, the plates are fixed at end A and rigidly connected at end B. How much do the metal plates expand in the direction of the length due to the increase in temperature T ?
 - (3) What are the thermal stresses σ_1 and σ_2 in metals 1 and 2 in question (2)?

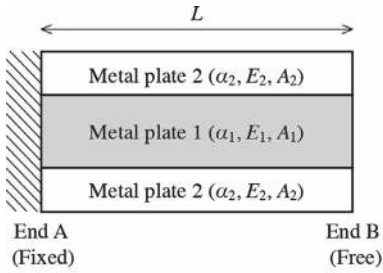


Figure 1

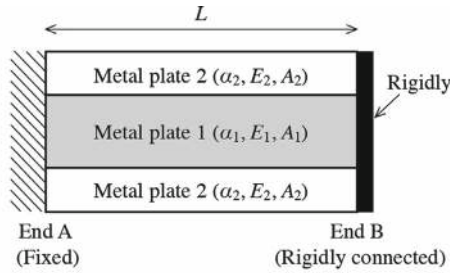


Figure 2

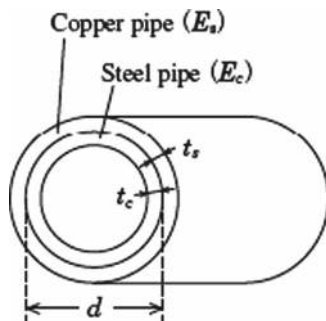
9. Answer the following questions.

- (1) Consider a thin spherical shell with an inner diameter d and a thickness t . Show that when an internal pressure P is applied to it, the increase in volume ΔV is given by the following equation:

$$\Delta V = \frac{\pi}{8} \cdot \frac{(1-\nu)}{E} \cdot \frac{d^3 P}{t}$$

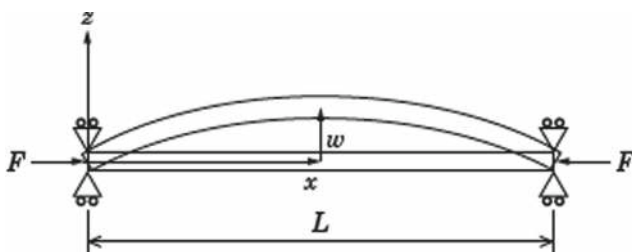
where E is the Young's modulus of the material of the shell and ν is the Poisson's ratio of the shell.

- (2) Consider a stainless steel spherical shell with an inner diameter $d = 2.5$ m and a thickness $t = 25$ mm, filled with water and closed at room temperature. Estimate the pressure in the water and the stress in the sphere when the temperature of the shell and the water increases by 20°C uniformly. Let E and ν be the Young's modulus and Poisson's ratio of the stainless steel, respectively, the bulk elastic modulus of water $K = 2.06$ GPa, the coefficient of linear expansion of the stainless steel $\alpha = 1.8 \times 10^{-5}/^\circ\text{C}$, and the coefficient of volume expansion of water $\beta = 3 \times 10^{-4}/^\circ\text{C}$.
10. Consider that as shown in the figure, a copper inner pipe is fitted in a steel outer pipe at room temperature T_0 . Derive equations giving the stresses in the inner and outer pipes when the temperature of the combined pipe increases to T . Assume that the stresses in both pipes at room temperature T_0 can be ignored. Let d be the inner diameter of the steel pipe as well as the outer diameter of the copper pipe, t_s be the thickness of the steel pipe, t_c be the thickness of the copper pipe, E_s be the Young's modulus of the steel pipe, and E_c be the Young's modulus of the copper pipe. Assume that both pipes are equal in length, and t_s and t_c are sufficiently small compared to d .



Buckling Problem

11. Consider a simply supported straight slender pipe with a length L , subjected to an axial compressive load F , as shown in the figure. Answer the following questions. As shown in the figure below, treat the pipe as a slender bar with Young's modulus E and a moment of inertia I that can rotate at both ends.
- (1) Let w be the deflection in the z -direction at a section x . Assume that w is sufficiently small. Derive the relationship between F and w .
 - (2) Obtain the general solution for w that meets relationship (1).
 - (3) Obtain the solution that meets relationship (1) and the boundary conditions $w(0) = w(L) = 0$.
 - (4) Obtain the buckling load.
 - (5) Obtain the shape of deflection at buckling.



Beam Problem

12. Consider that a rectangular straight hollow pipe, with a side length h , a wall thickness t , a length L , and Young's modulus E , as shown in Fig. 1, is simply supported at both ends and subjected to a load P at a distance $L/3$ from the left end and a load $2P$ at a distance $L/3$ from the right end. Answer the following

questions. Due to $L \gg h$, the pipe can be treated as a simply supported beam. Let I be the moment of inertia.

- (1) Obtain the shear stress and bending moment diagrams of the pipe.
- (2) Obtain the deflection curve of the pipe and then the maximum deflection.
- (3) Obtain the maximum bending stress in the pipe.

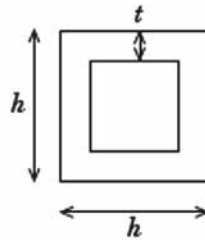


Figure 1

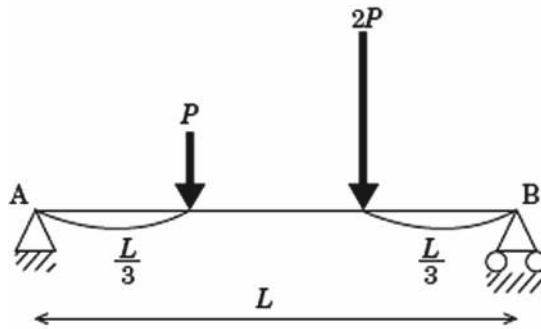


Figure 2

References¹

1. Arimitsu T (1992) Easy-to-understand introduction to strength of materials. Gijutsu Hyohron
2. Tsuji T (1992) Easy-to-Understand Strength of Materials. Kodansha
3. Yoshiki M, Kanazawa T (1959) Concise strength of materials. Yokendo
4. Oda J, Miyoshi T (1982) Strength of materials: drills. Saiensu-sha
5. Japan Society of Mechanical Engineers (1993) Mechanical engineering handbook: strength of materials. Maruzen, 1993

¹ (The following introductory level books are recommended to those who wish to study strength of materials and structural mechanics.)

There are many textbooks on strength of materials and structural mechanics. The following textbooks are commonly used.

The following are books recommended to those who are interested in the principle of energy.

6. Asada Y, Koibuchi K (1984) Comprehensive strength of materials, vol 8: mechanical structural strength. Ohmsha
7. Yoshida F. Basics of elastic and plastic mechanics. Kyoritsu Shuppan
8. Japan Society of Mechanical Engineers (1993) Mechanical engineering handbook: strength of materials. Maruzen
9. Landau LD, Lifshitz EM (1958) Mechanics (translated by Mito I, Hiroshige T). Tokyo Tosho
10. Landau LD, Lifshitz EM (1978) The classical theory of fields (translated by Tsuneto T, Hiroshige T). Tokyo Tosho

Chapter 3

Material Strength and Structural Strength



In order to achieve a structural design that adequately ensures the safe operation as well as the integrity of reactor components, their failure modes and mechanisms need to be understood. This chapter will cover the fundamentals of strength of materials and structural mechanics involved in the failure modes that are considered in design standards. Such basics knowledge will prepare for a deeper understanding of structural design methods, covered from Chap. 5. In addition to elastic behavior of materials, nonlinear behaviors such as plasticity and creep are to be considered when evaluating the cause of failure. Failure modes of a structure depend on three main components: the material properties, the shape of the structure, and the characteristic of the loads.

3.1 Deformation of Material

3.1.1 Elastic Deformation

As covered in Chap. 2, a linear relationship links stress and strain in elastic bodies. When stress is removed, strain becomes null and the structure recovers its original state. In a structure, stress and strain are represented by a tensor with multiple components. Since the components interact with each other through Poisson's ratio ν , the stress-strain relationship is represented by a full matrix. Stress and strain in a general three-dimensional problem are second-order tensors (3×3 components), and the elastic modulus is a fourth-order tensor ($3 \times 3 \times 3 \times 3$ components). Six of these components are independent of each other due to the compatibility conditions to be met by stress and strain. Instead of describing the stress-strain relationship component by component (a tedious task), the vector-matrix representation is often adopted to represent the stress-strain relationships in a single row. Stress and strain are then represented by vectors (first-order tensors) and the elastic modulus by a matrix (a

second-order tensor). Then, the linear stress–strain relationship (generalized Hooke’s law) in the elastic range can be given by Eq. (3.2) similar to Eq. (3.1).

For one-dimensional problems,

$$\sigma = E\varepsilon \quad (3.1)$$

For three-dimensional problems,

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xy} \\ \tau_{yz} \\ \tau_{zx} \end{Bmatrix} = \frac{E}{1+\nu} \begin{bmatrix} \frac{1-\nu}{1-2\nu} & \frac{\nu}{1-2\nu} & \frac{\nu}{1-2\nu} & 0 & 0 & 0 \\ \frac{\nu}{1-2\nu} & \frac{1-\nu}{1-2\nu} & \frac{\nu}{1-2\nu} & 0 & 0 & 0 \\ \frac{\nu}{1-2\nu} & \frac{\nu}{1-2\nu} & \frac{1-\nu}{1-2\nu} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{2} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{2} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1}{2} \end{bmatrix} \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \\ \gamma_{yz} \\ \gamma_{zx} \end{Bmatrix} \quad (3.2)$$

Equation (3.2) can be further simplified to $\{\sigma\} = [E] \times \{\varepsilon\}$. Such matrix and vector representation of the stress–strain relationship simplifies its description on paper, and becomes useful when developing computer programs.

Elastic deformation is characterized by the linear stress–strain relationship, even in multidimensions as shown in Eq. (3.2). Assuming elastic deformations, we can perform a theoretical analysis of relatively simple shapes. Numerical analyses can also be computed easily and quickly. When design calculations require to combine several loads due to specific conditions, stresses can actually be calculated independently for each load and linearly added afterward to obtain the total stress (superposition principle of linearity). This calculation process simplifies the original problem quite drastically, and therefore normally elastic analysis is often used in structural design.

In the following paragraph, basics of microscopic mechanism in materials (Fig. 3.1) will be discussed. Elastic deformation occurs due to the expansion and contraction of interatomic spacing in materials. Stress and strain components are coupled by the Poisson’s ratio because expansions and contractions of interatomic spacing generate changes in volumes. Most metals have a Poisson’s ratio of about 0.3. The interatomic spacing is restored to its original state when the load is removed.

[Supplement: Description of a Multiaxial Stress Fields Using the Summation Convention for Stress and Strain]

The summation convention is used to concisely describe tensors such as stress and strain in a multidimensional stress field. The suffix i to a variable indicates the range of i . If a suffix occurs twice in the same term, the summation is taken over the suffix. An index occurring only once is called a free index, and the summation is not taken over the index. See the example below:

$$\sigma_{jj} = \sigma_{11} + \sigma_{22} + \sigma_{33} \quad (j = 1, 2, 3) \quad (3.3)$$

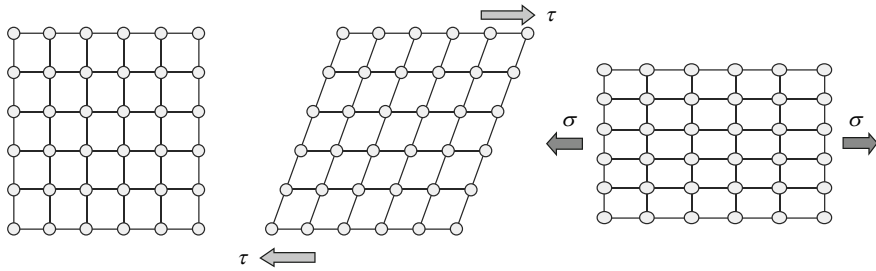


Fig. 3.1 Microscopic mechanism of elastic deformation

$$\begin{aligned} \sigma_{ij}\sigma_{ij} = & \sigma_{11}\sigma_{11} + \sigma_{12}\sigma_{12} + \sigma_{13}\sigma_{13} + \sigma_{21}\sigma_{21} + \sigma_{22}\sigma_{22} + \sigma_{23}\sigma_{23} \\ & + \sigma_{31}\sigma_{31} + \sigma_{32}\sigma_{32} + \sigma_{33}\sigma_{33} \quad (i, j = 1, 2, 3) \end{aligned} \quad (3.4)$$

3.1.2 Plastic Deformation

[1] Uniaxial Stress–Strain Curve

Figure 3.2 shows a typical uniaxial stress–strain curve for a highly ductile metal such as austenitic stainless steel. As shown in Fig. 3.2a, when a uniaxial tensile load is applied, the stress σ increases proportionally to strain ε , to point A. This relationship is given by $\sigma = E\varepsilon$ using Young's modulus E (Hooke's law). The stress σ_y at point A is called the yield stress. Plastic deformation ε^p begins to occur beyond point A, and the stress gradually increases due to work hardening. When the load is reduced at point B (unloading), the stress decreases from B to C in parallel with OA, and plastic strain ε^p represents the remaining strain at zero stress (OC). CD is elastic strain ε^e , which is proportional to the stress at point B. The total strain ε at point B is the sum of the elastic strain ε^e and the plastic strain ε^p .

$$\varepsilon = \varepsilon^e + \varepsilon^p = \sigma/E + \varepsilon^p \quad (\sigma > \sigma_y) \quad (3.5)$$

As shown in Fig. 3.2b, when the loading is reduced from point B, and furthermore a compressive load is applied to the structure, the stress–strain relationship changes as described by BCD, and the material yields at a compressive stress (point C) smaller in absolute value than the yield stress (point A) for the initial tensile loading. This phenomenon is called Bauschinger's effect. When the strain is returned to zero (point D), a certain amount of stress remains. Such stress is called residual stress.

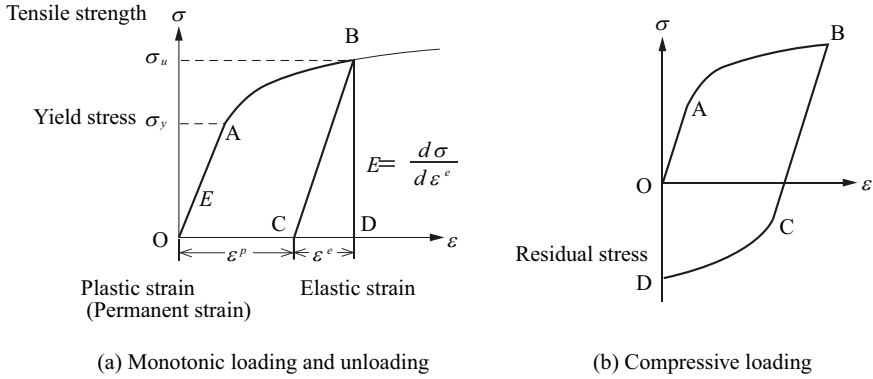


Fig. 3.2 Uniaxial stress–strain curve

In the elastic region, stress is proportional to strain, but this proportionality does not hold in the plastic region. The plastic strain at a certain point depends on the stress at that point as well as the history of the stress applied up to that point. The incremental theory of plasticity is commonly used in theories that take this dependence to loading history into account. On the other hand, the total strain theory is a simplified theory that does not consider the history of stress, and is therefore used as an approximated method.

As described above, stress–strain relationships for real materials are complex, making it difficult to be used in design and analysis without simplification. Therefore, simplified models are commonly used. A typical example of such model is an elastic–perfectly plastic model shown in Fig. 3.3. This model ignores work hardening and assumes a constant stress after the yield stress. This model simplifies the stress–strain relationship to only two parameters—Young’s modulus E and the yield stress σ_y . However, it usually produces conservative results as it assumes a softer material than actual one. Since conservativeness relates to safer designs, such simplified model is commonly used in design evaluation. Another approximation model, the bilinear model, characterizes the plasticity zone of the stress–strain curve with a constant work-hardening coefficient H' .

$$H' = d\sigma/d\epsilon^p \quad (3.6)$$

Using Eqs. (3.5) and (3.6), the slope of the curve after the yield point in the bilinear approximation model is given by

$$d\sigma/d\epsilon = EH'/(E + H') \quad (3.7)$$

For a same material, the work-hardening coefficient H' can vary depending on the strain range considered. The bilinear approximation model allows a more accurate

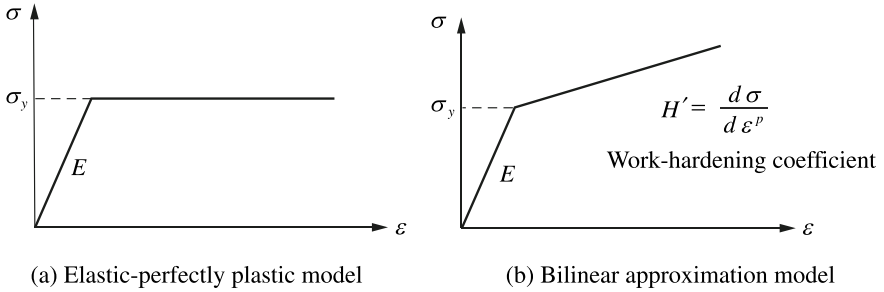


Fig. 3.3 Stress–strain curve models used in design

analysis than the elastic-perfectly plastic model when the slope coefficient is properly specified. It is therefore commonly adopted in elastic–plastic analyses using the finite element method [1].

[2] Yield Criterion

The yield criterion corresponds to the stress limit when the material deformation becomes plastic (instead of elastic). Equivalent stress is used to define the yield criteria in a multidimensional stress field (or a multiaxial stress field). The yield criterion in a multiaxial stress state (tensor) is described by an equivalent stress which is related to the yield stresses in a uniaxial stress state (scalar) (Fig. 3.4). Two criteria are commonly used: the Mises and Tresca criteria. The Mises yield criterion is given by

$$f = \bar{\sigma}_{\text{Mises}}^2 - \sigma_y^2 = 0 \tag{3.8}$$

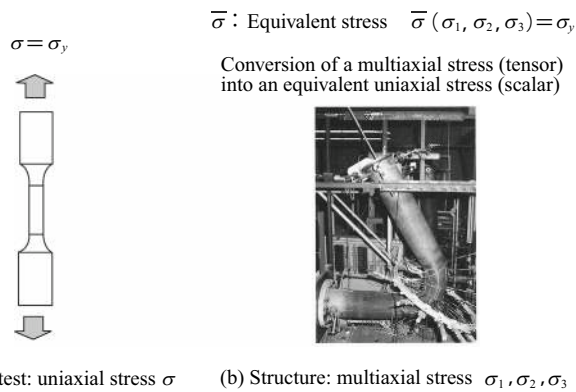


Fig. 3.4 Yielding of a material and the yield criteria for materials and structures

where,

$$\begin{aligned}
 \bar{\sigma}_{\text{Mises}} &= \sqrt{\frac{3}{2} \sigma'_{ij} \sigma'_{ij}} \\
 &= \sqrt{\frac{1}{2} [(\sigma_x - \sigma_y)^2 + (\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_x)^2 + 6(\tau_{xy}^2 + \tau_{yz}^2 + \tau_{zx}^2)]} \\
 &= \sqrt{\frac{1}{2} [(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]} \quad (3.9)
 \end{aligned}$$

σ_{Mises} is called the Mises equivalent stress. σ_1 , σ_2 , and σ_3 are principle stresses. The Mises yield criterion is equivalent to that when the shear strain energy reaches a critical value, the material yields, and is called the shear strain energy theory. Read the supplement for the details of equivalent stress. $f = 0$ indicates a yield state and, $f < 0$ indicates an elastic state.

The Tresca yield criterion is given by

$$\begin{aligned}
 f &= \sigma_{\text{Tresca}} - 2\tau_0 = 0 \\
 \sigma_{\text{Tresca}} &= \text{Max}[|\sigma_1 - \sigma_2|, |\sigma_2 - \sigma_3|, |\sigma_3 - \sigma_1|] \quad (3.10)
 \end{aligned}$$

where σ_{Tresca} is called the Tresca equivalent stress, which is twice the maximum shear stress, and τ_0 is the shear yield stress. The Tresca yield criterion is also called the maximum shear stress theory and is equivalent to the following: the material yields when the maximum shear stress reaches the shear yield stress.

In Fig. 3.5, the yield criterion is represented by a curved surface in a stress space, which is called the yield surface.

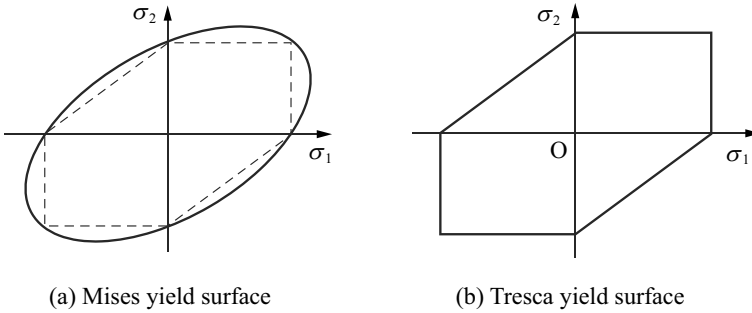


Fig. 3.5 Yield surface in a multiaxial stress state (for $\sigma_3 = 0$)

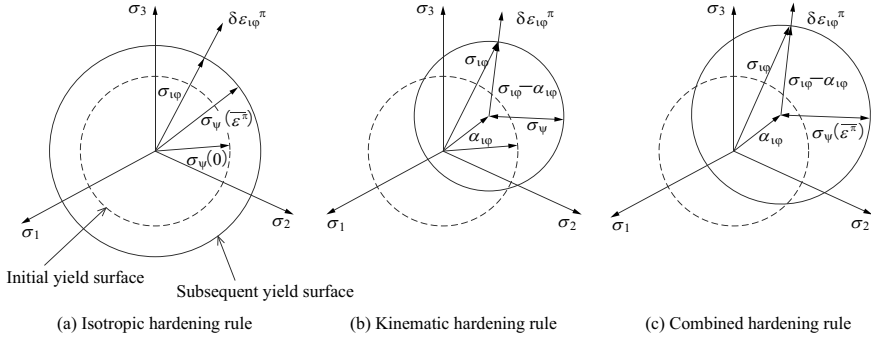


Fig. 3.6 Basic hardening rules

[3] Hardening Rules

In order to describe plastic behavior under varying stress, rules (hardening rules) are necessary to describe how the yield surface changes with increasing plastic deformation. There are two basic hardening rules: the isotropic hardening rule and the kinematic hardening rule.

The isotropic hardening rule is a model in which the center of the yield surface is fixed, while the size of the surface increases along with the plastic deformation, as represented in Fig. 3.6a. This rule cannot describe Bauschinger's effect but can properly describe hardening behavior under nearly monotonic loading. It also makes analyses easier and is therefore commonly used for analysis by the finite element method (FEM). The yield function in the isotropic hardening rule is given by

$$f = \frac{3}{2} \sigma'_{ij} \sigma'_{ij} - [\sigma_y(\bar{\epsilon}_p)]^2 = 0$$

$$\begin{aligned} \bar{\epsilon}_p &= \sqrt{\frac{2}{3} \epsilon_{ij}^p \epsilon_{ij}^p} = \sqrt{\frac{2}{3} \sqrt{\epsilon_x^{p2} + \epsilon_y^{p2} + \epsilon_z^{p2} + \frac{1}{2} (\gamma_{xy}^{p2} + \gamma_{yz}^{p2} + \gamma_{zx}^{p2})}} \\ &= \frac{\sqrt{2}}{3} \sqrt{(\epsilon_x^p - \epsilon_y^p)^2 + (\epsilon_y^p - \epsilon_z^p)^2 + (\epsilon_z^p - \epsilon_x^p)^2 + \frac{3}{2} (\gamma_{xy}^{p2} + \gamma_{yz}^{p2} + \gamma_{zx}^{p2})} \end{aligned} \quad (3.11)$$

where $\bar{\epsilon}^p$ is called the Mises equivalent plastic strain.

The kinematic hardening rule is a model where the size of the yield surface remains unchanged and the location of the center of the surface moves with increasing plastic deformation, as described in Fig. 3.6b. This rule can describe Bauschinger's effect and therefore properly describes hardening behavior under cyclic loading. It is also commonly used for FEM analyses. The yield function in the kinematic hardening rule is given by

$$f = \frac{3}{2}(\sigma'_{ij} - \alpha_{ij})(\sigma'_{ij} - \alpha_{ij}) - \sigma_y^2 = 0 \quad (3.12)$$

where α_{ij} is the displacement of the center of the yield surface and is called the back stress.

Another hardening rule, called the combined hardening rule, considers both isotropic and kinematic hardening rules, as represented in Fig. 3.6c. This rule is given by

$$f = \frac{3}{2}(\sigma'_{ij} - \alpha_{ij})(\sigma'_{ij} - \alpha_{ij}) - [\sigma_y(\bar{\varepsilon}_p)]^2 = 0 \quad (3.13)$$

Other detailed hardening rules have been studied and developed in order to better describe hardening behavior when large changes in loading direction are repeated many times.

[4] Flow Rules

The variation rate of plastic strain $\{\varepsilon^p\}$ and stress $\{\sigma\}$ in a multiaxial stress field can be explicitly formulated by defining a plastic potential and a flow rule, in addition to the yield criterion and the hardening rule. Generally, the Mises yield condition equation f is used to describe a plastic potential, associated to the flow rule described by the following equation.

$$d\varepsilon_{ij}^p = \frac{\partial f}{\partial \sigma_{ij}} d\lambda \quad (d\lambda \text{ is a positive scalar function}) \quad (3.14)$$

Finally, microscopic mechanism of plastic deformation is briefly described in the following paragraph. In lattice arrangement of metals, dislocations are natural defects (namely imperfections) that locally cause very high stress states. When the load applied is inferior to a specific critical value or when stress is below the critical shear stress, the dislocations cannot move inside the lattice. When such critical value is exceeded, the lattices adjacent to the dislocation are displaced, causing a propagation of the dislocations, resulting in macroscopic deformations. Figure 3.7 schematically describes such phenomenon, which is called plastic deformation and is irreversible. When one dislocation moves (slips), an interference with the movement of other dislocations occurs. As a result, the load must be increased to cause further deformation, a phenomenon called work hardening. As the movement of dislocations is a slip, and therefore there is no volume change. With increasing elastic-plastic deformation, the apparent Poisson's ratio approaches 0.5 from about 0.3 in elastic state.

[Supplement: Yield Criterion and Equivalent Stress in a Multiaxial Stress Field]

The method to define yield in a multiaxial stress state (represented by a tensor) is briefly described below. The yield criterion for isotropic materials must be independent from any coordinate transformation and is therefore expressed as follows, in

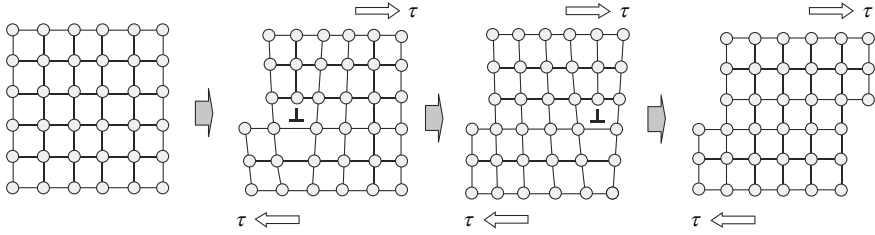


Fig. 3.7 Microscopic mechanism of plastic deformation

terms of the stresses J_1 , J_2 , and J_3 , invariant to the coordinate system.

$$f = f(J_1, J_2, J_3) = 0 \quad (3.15)$$

where,

$$\begin{cases} J_1 = \sigma_x + \sigma_y + \sigma_z \\ J_2 = (\sigma_x\sigma_y + \sigma_y\sigma_z + \sigma_z\sigma_x) + \tau_{xy}^2 + \tau_{yz}^2 + \tau_{zx}^2 \\ J_3 = \sigma_x\sigma_y\sigma_z - \sigma_x\tau_{yz}^2 - \sigma_y\tau_{zx}^2 - \sigma_z\tau_{xy}^2 + 2\tau_{xy}\tau_{yz}\tau_{zx} \end{cases} \quad (3.16)$$

The mean value of normal stresses obtained from J_1 is called the hydrostatic stress and is given by

$$\sigma_m = J_1/3 \quad (3.17)$$

For metals, as a first approximation, yielding can be considered independent from the hydrostatic stress. Then, Eq. (3.15) can be written as

$$f = f(J'_2, J'_3) = 0 \quad (3.18)$$

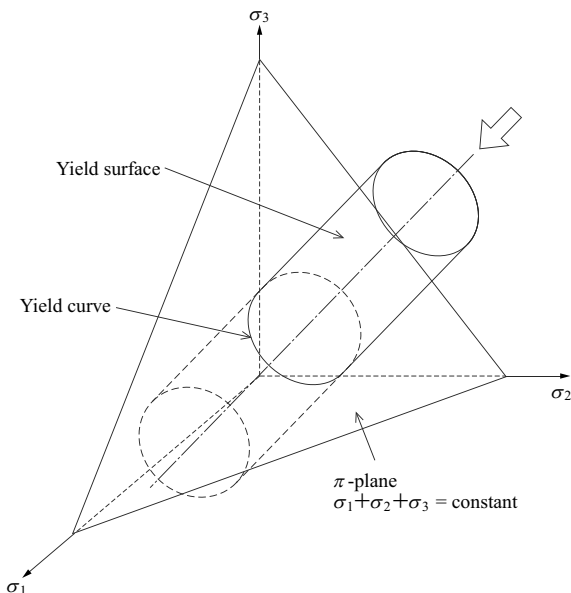
where J'_1 , J'_2 and J'_3 are invariant to the deviatoric stress σ_{ij} given by

$$\sigma'_{ij} = \sigma_{ij} - \sigma_m \delta_{ij} \quad (\text{for } i = j, \delta_{ij} = 1; \text{ for } i \neq j, \delta_{ij} = 0) \quad (3.19)$$

One solution that meets Eq. (3.18) is the Mises yield criterion from Eq. (3.8).

The following paragraph will describe the methods which allow geometric representation of stress. A stress state can be defined by the value of its three principal stress components. Therefore, the yield criterion is commonly expressed in a three-dimensional space with the principal stresses defined in Cartesian coordinates, as described in Fig. 3.8. In this case, let's note that the yielding is independent from the hydrostatic stress. In addition, we can describe the characteristics of the yield criterion by the yield curve which appears on the cutting plane with constant hydrostatic stress (called the π -plane). The direction cosine of the π -plane is $(1/\sqrt{3}, 1/\sqrt{3}, 1/\sqrt{3})$. The yield curve is the same on all sections where the hydrostatic stress

Fig. 3.8 Geometric representation of the yield condition



is constant. Then, by placing all yield curves in a three-dimensional space, a yield surface can be created. It is normal to the π -plane and has uniform cross sections. Figure 3.8 describes the yield curves in a three-dimensional space with the principal stresses defined as the Cartesian coordinates and where the π -plane is viewed from the direction normal to the section planes (outlined by the arrow).

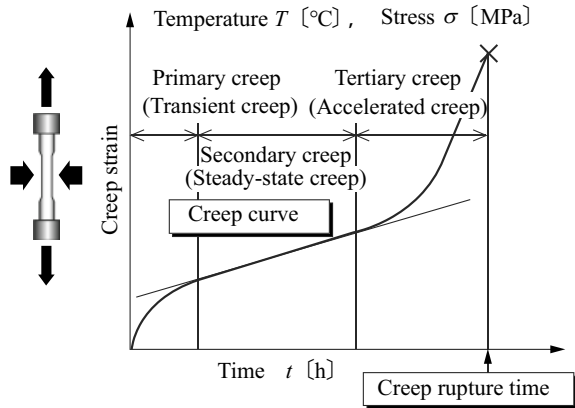
3.1.3 Creep Deformation

[1] Uniaxial Creep Strain Equation

At a temperature higher than a certain level, dislocations are thermally activated by stress levels even below the critical shear stress, and thus deformation occurs with loads inferior to the critical load discussed in the previous paragraph. Such deformation is called “creep deformation.” Creep deformation increases with time under constant stress (loading). Due to high-temperature levels, the structural design of a fast reactor (described in Chap. 6) requires considering creep deformation when evaluating its design. The relationship between the creep strain ε^c and time t is derived experimentally by applying a constant tensile stress σ at a constant temperature in creep testing is called a creep curve. Figure 3.9 is a typical example of a creep curve. Generally, the creep curve can be divided into three stages:

- (1) Primary creep (transient creep): The initial stage of creep. The creep rate decreases with time, mainly due to the development of dislocation structures.

Fig. 3.9 Creep deformation



- (2) Secondary creep (steady-state creep): In this stage, the strain rate is nearly constant. This constant strain rate is called the steady-state creep rate or the minimum creep rate. Hardening and recovery mechanisms at high temperatures are balanced, resulting in a constant creep rate.
- (3) Tertiary creep (accelerated creep): In this stage, the strain rate increases gradually, leading to rupture. Such phenomenon can be explained by the accumulation of voids in the structure, thereby creating a decrease in the substantial cross-sectional area.

Figure 3.10a shows the increase of creep strain along with increasing stress at a constant temperature. Figure 3.10b describes the increase of creep strain at constant stress, but rising temperature.

Next, the following models and equations are commonly used to describe the creep deformation phenomenon:

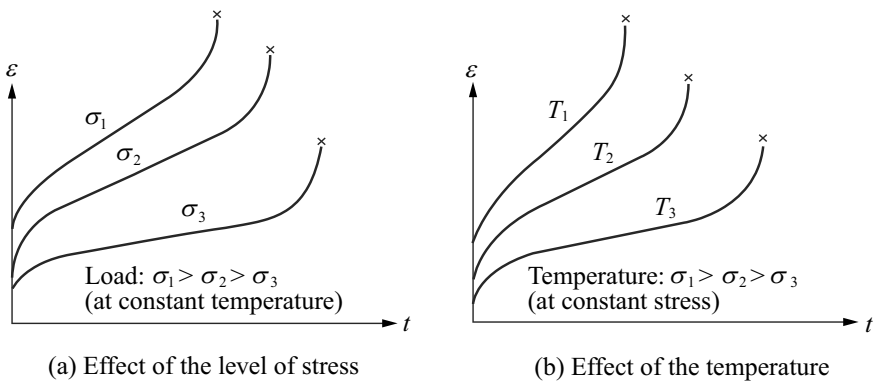


Fig. 3.10 Effect of stress and temperature on the creep curve

- Norton's rule

$$\dot{\varepsilon}_c = A\sigma^n \quad (3.20)$$

Norton's rule is the simplest equation to describe the creep strain rate. A and n are material- and temperature-dependent parameters. In general, material used in reactors structure possesses an n value larger than 5, with a very strong nonlinearity of creep. Therefore, creep strain in such type of structures tends to occur selectively in the high-stress regions.

- Norton-Bailey equation

$$\varepsilon_c = k\sigma^n t^m \quad (3.21)$$

where t is time, and k , n , and m are material- and temperature-dependent parameters. This equation describes the temporal change in creep strain ε^c , to some extent.

- Blackburn type strain equation

$$\varepsilon_c = C_1(1 - e^{-r_1 t}) + C_2(1 - e^{-r_2 t}) + \dot{\varepsilon}_m t \quad (3.22)$$

The first and second terms on the right-hand side of the equation represent primary creep and the third term represents secondary creep. Each parameter C_1 , C_2 , r_1 , and r_2 is a function of stress and temperature.

Without plastic strain ε^p , the total strain ε is the sum of the elastic strain ε^e and the creep strain ε^c as shown in the following equation.

Under a constant total strain, an increase in creep strain makes the elastic strain decrease, therefore reducing the stress. Such process is called stress relaxation.

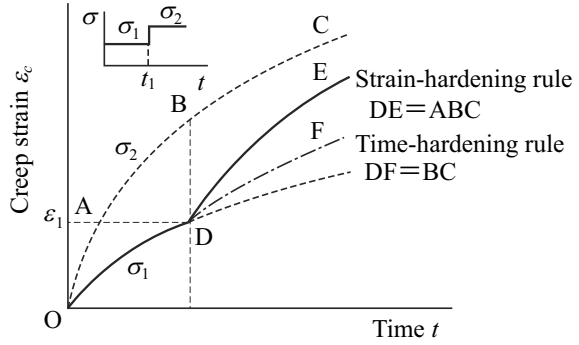
$$\varepsilon = \varepsilon^e + \varepsilon^c = \sigma/E + \varepsilon^c \quad (3.23)$$

[2] Hardening Rules

In actual structures, the stress and the temperature change depending on the loading and boundary conditions. Therefore, a creep hardening rule must be defined to consider their history. Typical creep hardening rules are: the time-hardening rule and the strain-hardening rule.

The time-hardening rule is a model which determines the creep rate from the three parameters: the current temperature, the stress, and the duration after initiation of creep. The creep rate is independent from the history of loading. This rule tends to underestimate the creep rate for monotonically increasing stress and overestimate it for monotonically decreasing stress. Applying the time-hardening rule to Eq. (3.21), we obtain

Fig. 3.11 Basic creep hardening rules



$$\dot{\varepsilon}_c = mk\sigma^n t^{m-1} \quad (3.24)$$

The strain-hardening rule is a model which determines the creep rate from the cumulative creep strain and the current temperature. The creep rate thus depends on the history of loading. This rule describes adequately phenomena in which stress and temperature vary. It is also commonly used for FEM analysis. In the strain-hardening rule, the creep rate is derived by eliminating time t in Eqs. (3.21) and (3.24) as shown in Eq. (3.25). Figure 3.11 illustrates the difference between the time-hardening rule and the strain-hardening rule when stress varies stepwise.

$$\dot{\varepsilon}_c = mk^{\frac{1}{m}} \sigma^{\frac{n}{m}} \varepsilon_c^{\frac{m-1}{m}} \quad (3.25)$$

[3] Flow Rules

Creep strain in a multiaxial stress state can be described in a similar manner to plastic strain by using the Mises equivalent stress [Eq. (3.9)], while replacing the stress and the strain of the uniaxial creep strain equation by the equivalent creep strain ε^c and the equivalent creep rate $\dot{\varepsilon}^c$ given by

$$\overline{\varepsilon}_c = \sqrt{\frac{2}{3} \varepsilon_{ij}^c \varepsilon_{ij}^c}, \quad \overline{\dot{\varepsilon}}_c = \sqrt{\frac{2}{3} \dot{\varepsilon}_{ij}^c \dot{\varepsilon}_{ij}^c} \quad (3.26)$$

In addition, the relationship between creep rate $\{\dot{\varepsilon}^c\}$ and stress $\{\sigma\}$ in a multiaxial stress field can be explicitly formulated by defining the Mises type creep potential and its associated flow rule.

$$\dot{\varepsilon}_{ij}^c = \frac{3}{2} \frac{\overline{\dot{\varepsilon}}_c}{\overline{\sigma}} \sigma'_{ij} \quad (3.27)$$

The equation above implies that the creep strain rate components occur in the normal direction to the creep potential, which is also the direction of the deviatoric stress.

[Supplement: Mechanism of Creep Deformation]

In presence of an external force, dislocations move relatively easily, causing plastic deformation. In addition, when the crystal is heated, dislocations tend to move more freely due to the thermal vibration of atoms around the equilibrium point. At high temperatures, the thermal vibration is strong, and a time-constant external force causes progressive deformation. This phenomenon is called creep. The thermal vibrations cause statistically irregular stress variations. The average number of stress peaks at energy A per unit volume and unit time is given by

$$\nu = Be^{-\frac{A}{kT}} \quad (3.28)$$

where B is a constant, T is the absolute temperature, and k is the Boltzmann constant. Then, using the constant D , the average creep rate can be written as a function of temperature:

$$\dot{\epsilon} = De^{-\frac{c}{kT}} \quad (3.29)$$

3.2 Fracture Mechanisms of Materials

3.2.1 Brittle Fracture and Ductile Fracture

The fracture mode of a metal subjected to a monotonic tensile load differs depending on the type of the metal, and its temperature. There are two fracture modes, brittle and ductile fracture:

- Brittle fracture: involves little plastic deformation before fracture
- Ductile fracture: involves significant plastic deformation before fracture.

In brittle fracture, the material fractures suddenly without deformation. For instance, glass is a well-known brittle material. Consequently, brittle fracture is more dangerous than ductile fracture when it occurs in structures. Brittle materials such as cast iron and high-strength steel are susceptible to brittle fracture, while ductile materials such as stainless steel have a lower susceptibility (to break suddenly). Such susceptibility to brittle fracture becomes higher at lower temperatures. In reactors, a well-known phenomenon causes metal to become more susceptible to brittle fracture after being irradiated for a certain amount of time.

The Charpy impact test aims at assessing the fracture behavior of steels. In this test, the specimen is impacted to fracture by a hammer at various temperatures to measure the energy required for fracture, or the energy absorbed by the specimen until it fractures. Figure 3.12 describes the Charpy impact test and an example of test results. The fracture mode changes from brittle to ductile fracture depending on the temperature. Fractures at lower temperature occur at lower absorbed energy,

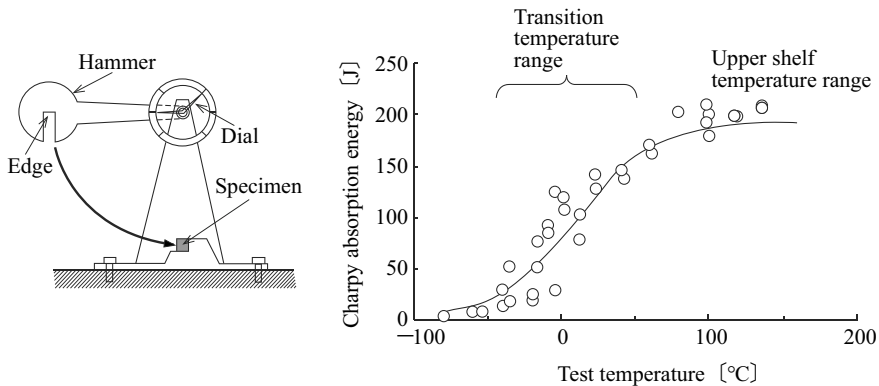


Fig. 3.12 Charpy impact test and example of test results

whereas fractures at higher temperatures require higher absorbed energy. As shown in the figure, the temperature range which corresponds to a large variation in energy is called the *ductile–brittle transition temperature* (DBTT) (http://en.wikipedia.org/wiki/Ductile-brittle_transition_temperature) range. In the higher-temperature range, the absorption energy becomes nearly constant, and such temperature range is called the upper shelf temperature range.

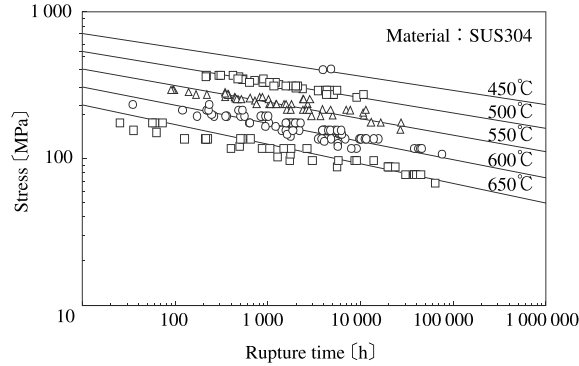
Carbon and low-alloy steels used for reactor components are susceptible to brittle fracture at such low temperatures. To prevent brittle fracture, temperature ranges and operating temperature limits are defined and specified in codes and standards.

3.2.2 Creep Rupture

As shown in Fig. 3.9, creep deformation increases even under constant stress. If it continues to increase, the material will eventually fracture. Therefore, the creep strength of the material is defined by the creep rupture time until fracture occurs under constant stress. Similarly to creep strain (Fig. 3.10), creep rupture time depends on stress and temperature. For example, Fig. 3.13 shows the creep rupture time t_R of 304SS steel (stainless steel) at various stresses and temperatures in the stress- t_R diagram. This figure indicates that the creep rupture time monotonically decreases with increasing stress, at a constant temperature. The creep rupture time is generally shorter at higher temperatures. In the figure, the stress- t_R curve has the tendency of shifting downward.

As shown in Fig. 3.13, the creep rupture time is described as a function of stress, which depends on temperature. However, similarly to creep strain, it is more convenient to describe the creep rupture time as a function of both temperature and stress in a unified manner. Several approaches have been proposed for this purpose. A typical one is the Larson-Miller time-temperature parameter (LMP) given by

Fig. 3.13 Relationship among the creep rupture time, temperature and stress



$$\text{LMP} = T(\log t_R + C) = h(\sigma) \tag{3.30}$$

where C is a constant. The equation implies that $h(\sigma)$ is a function of stress only. If phenomena are governed by the same mechanism, then creep rupture data obtained at different temperatures can be evaluated in a unified manner by the Larson-Miller parameter. For example, Fig. 3.14 plots the data in Fig. 3.13 after evaluation from the Eq. (3.30). Relatively long-term creep strength can be predicted from the short-term test data (accelerated-test data) at temperatures higher than operating temperatures, using the Larson-Miller parameter. However, it should be noted that if a change in structure or microscopic mechanism occurs due to a temperature increase, the material constant C requires to be reconsidered to perform calculations on data.

The microscopic mechanism of creep rupture has been described in Fig. 3.15. When long-term loads such as creep or creep-fatigue is applied at high temperatures, grain boundary slips occur and, as a result, voids and cavities are formed. The cavities

Fig. 3.14 Relationship between the Larson-Miller time-temperature parameter and stress

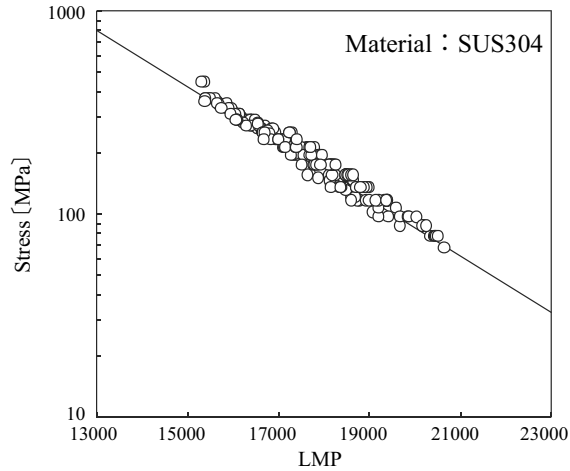
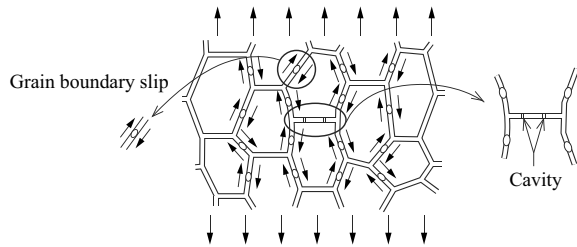


Fig. 3.15 Microscopic mechanism of creep rupture



increase and grow over time and combine to form microcracks and separate the grain boundary. If this process continues further, these microcracks grow into macrocracks, leading to rupture.

In order to predict long-term creep strength from accelerated, short-term creep rupture test data at higher temperatures, the rupture mechanism as described above should be the same in both short- and long-term time.

3.2.3 Fatigue and Creep-Fatigue

Let's consider a beam which is bent repeatedly over time (constant loading). Even though the bending load is too low to affect the structure on a monotonic loading, over a large number of cyclic loads, cracks have initiated on the surface of the beam, and propagated until the beam finally breaks. This type of failure mode is called fatigue. The fatigue strength of a material is defined by the number of displacement or loading cycles applied to the specimen until failure occurs, as illustrated in Fig. 3.16. If the range of the strain caused by the cyclic displacement is large, the material undergoes a plastic cycle (such as its initial cycle CDEF, in the figure) and the material fails at relatively low number of displacement cycles. If the range of the strain is small, the material undergoes elastic cycles (AB in the figure) and the number of displacement cycles to failure increases. The relationship between the strain range and the number of cycles to failure is shown in the right figure in Fig. 3.16. The strain range monotonically decreases with increasing cycles to failure, approaching a certain constant value. This relationship shows the fatigue strength of a material and therefore is called a fatigue curve. The fatigue in the range up to 10^5 or 10^6 cycles, which is affected by plastic strain, is called low-cycle fatigue. Fatigue in the range of higher cycles is called high-cycle fatigue.

Another typical failure mode of a structure subjected to cyclic loading is creep-fatigue failure. The fatigue life is significantly affected by the creep effect. Figure 3.17 illustrates a stress-strain hysteresis loop that can be formed when a trapezoidal wave loading is applied with the introduction of a hold time (duration of the tensile loading). Even though stress is relaxed due to creep during the hold time, the fatigue life is also reduced due to the damage generated by the creep phenomenon.

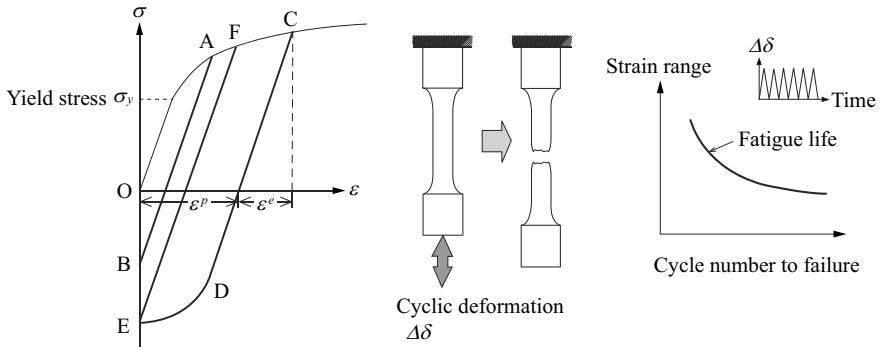


Fig. 3.16 Fatigue

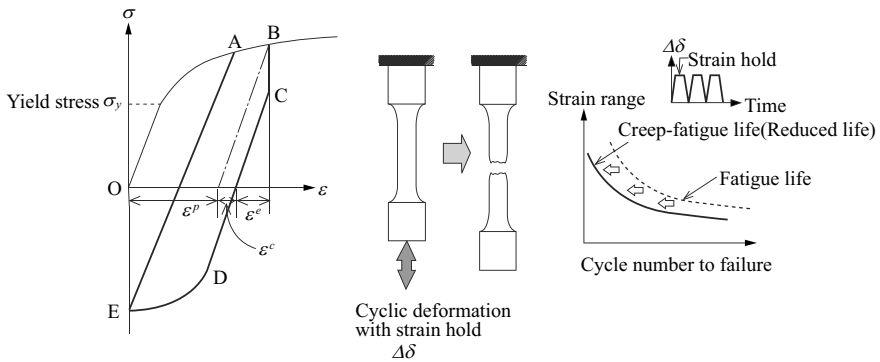


Fig. 3.17 Creep-fatigue

Figure 3.18a illustrates schematically the mechanism of fatigue damage. If the stress produced by the loading exceeds the yield strength value, the entire material is plastically deformed. As a result, new surfaces are exposed by slips, increasing the surface area and causing larger elongations. When the stress is below the yield strength, global plastic deformation (of the entire material) does not occur. Local slips can preferentially occur during large phases or when grains on the surface layer of the material are weak and favorably oriented for a slip. If such stress is repeated, the slip causes a shape change on the surface. The slip surface under cyclic stress thus becomes undulated. Once the undulated surface is formed, stress concentration occurs at this point. As a result, the concave region preferentially extends inward due to an irreversible slip, which finally becomes the initiation of fatigue cracks. A macroscopic crack or scratch on the material surface can also cause stress concentration, making the undulated surface even more susceptible to fatigue cracking. Once the cracks appear in the slip direction of a metal and grow to macroscopic dimensions, they propagate in the normal direction to the stress and eventually penetrate

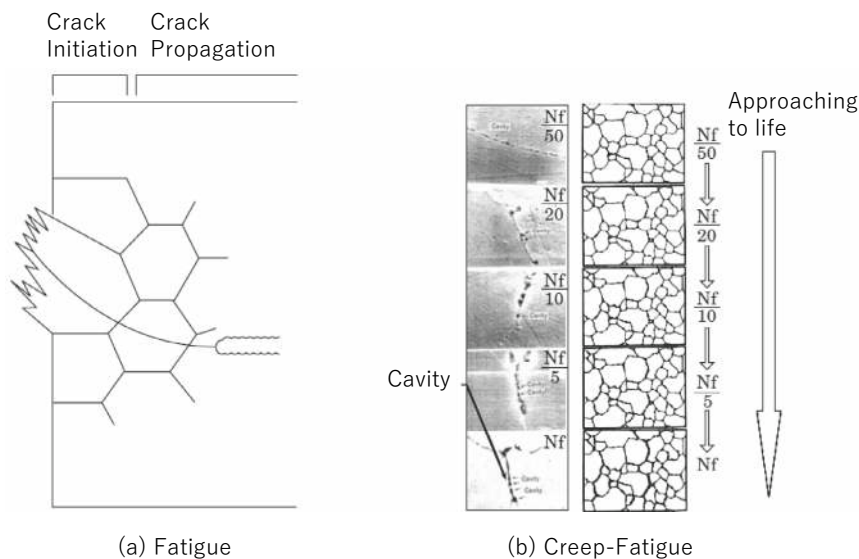


Fig. 3.18 Mechanism of fatigue crack and creep-fatigue crack initiation and propagation

the wall according to fracture mechanics, which laws are described in Chap. 10. In creep-fatigue, creep damage is mainly due to cavity formation in the grain boundary, as described in Fig. 3.15, and affects the structure, in addition to the effects generated by fatigue damage. As a result, the fatigue strength decreases due to the combined effect of creep and fatigue. In creep-fatigue, recombination of cavities, as well as separation of grain boundary, lead to larger damage than fatigue, as described by Fig. 3.18b.

3.3 Types of Loads and Response of Structures

3.3.1 Different Types of Loads

As described in Table 3.1, typical loads to be considered in nuclear power plants are internal pressure, dead weight, thermal stress, and seismic loads. They all possess different characteristics and therefore have specific effects regarding failure modes [2].

Loading can be classified into two types, depending on the stress–strain responses they induce: load-controlled and displacement-controlled types of loading. The concept of this classification, which is not generally familiar, is described below. In a structure, the main stress induced by an external force is called the primary stress. For instance, the stress generated in a beam with one end fixed and a weight placed

Table 3.1 Typical loads and their characteristics in nuclear power plants

Type of loads	Characteristic of the loading	Loading cycles	Loading duration
Internal pressure	Load-controlled	Low	Long
Dead weight	Load-controlled	Non	Long
Thermal stress	Displacement-controlled	High	Short
Seismic load	Load/displacement-controlled ^a	High	Short

^a Depends on the conditions

at the other end. In an elastic-perfect plastic body, once the primary stress reaches the yield strength of the material, plastic deformation continues to grow indefinitely, which means that plastic instability occurs without the additional external force. An example of such primary stress would be the hoop stress in a cylindrical vessel subjected to internal pressure, or the stress generated in a support structure bearing the weight of other components. A secondary stress is a component of the stress generated in a structure to primarily meet the compatibility condition of deformation. For example, it would be the stress induced in a bar with one end fixed and a certain amount of forced displacement applied to the other end. If this stress exceeds the yield stress, plastic deformation appears but does not grow indefinitely unless the external force is increased. In this case, so-called self-controlled deformation occurs. A practical example would be the thermal stress occurring in a pipe with both ends fixed, due to thermal expansion. Another example would be the discontinuity stress which occurs at the junction between the cylindrical and spherical shells of a pressure vessel subjected to internal pressure, in order to absorb the difference in radial displacement between them.

According to the characteristics presented in the previous paragraph, the primary stress is a load-controlled stress, and the secondary stress is a displacement-controlled stress. They have different effects on structural integrity. The peak stress described in Sect. 3.3.4 is displacement-controlled stress. This concept can be extended to the classification of loads acting on structures to define the difference in their effects on structural integrity. For example, once a load such as pressure or dead load reaches the plastic collapse limit of the structure, it collapses unless the load decreases. This loading can be called load-controlled loading. For instance, a thermal load, induced by thermal expansion, generates self-controlled stress in a structure and therefore cannot cause plastic collapse unless it increases indefinitely. Due to the definite relationship between the load and the displacement of the structure, such stress is displacement-controlled.

For constant or less cyclically applied loads such as internal pressure and dead weight, attention must be paid primarily to failures due to monotonic loading, such as fracture, collapse, and buckling. For cyclically applied loads (such as thermal stress induced by varying fluid temperature, or seismic loads), failure modes that are specific to cyclic loading, such as fatigue and creep-fatigue, must be considered. Generally, seismic loads have been treated as load-controlled loading. However,

recent experimental studies have demonstrated that this failure mode is displacement-controlled when considered under normal conditions.

The last topic in this section is the loading duration. Long-holding loads affect creep strength at high temperatures. Usually, thermal and seismic stresses are known to take effect for short periods of time. However, it should be noted that if the stress induced exceed the yield strength of the material, the residual stress could cause creep damage.

3.3.2 Load-Controlled Stress and Displacement-Controlled Stress

When load-controlled (primary) stress or displacement-controlled (secondary) stress is applied to a structure, it undergoes an elastic–plastic response. As an example of load-controlled stress, let's consider a bar with an increasing force applied at its end, as shown in Fig. 3.19. In addition, stress and strain are varying as the load is increased at a constant rate until time t_0 . Similarly, in a pressurized vessel at a constant rate, the hoop stress and strain in the shell relate in nature to the stress and strain in the bar. Axial normal stress occurs in the bar, which is determined by an equilibrium equation with the external force. Strain increases proportionally to stress according to the theory of elasticity, and rapidly increases due to plastic deformation when the stress reaches the yield stress. The temporal variation in the stress and strain in the bar and the stress–strain relationship that have been discussed above have been illustrated in the top-right figure in Fig. 3.19. The symbols in the figure correspond to the case dealing with the temporal variation.

As an example of displacement-controlled stress, let's consider a bar with a forced displacement applied at its end, as depicted in Fig. 3.19. In addition, the stress and strain are varying as displacement is increased at a constant rate until time t_0 . Let's also consider a vessel heated at a constant rate with its supporting structures fixed on the cold floor. The stress and strain in the supporting structures are similar in nature to the stress and strain in the bar except for the effect of the elastic follow-up, which will be described later. The strain in the bar is determined by geometric compatibility according to the amount of forced displacement and increases at a constant rate. The stress also increases in proportion to the elastic strain. Until the stress reaches the yield stress, the strain remains elastic, and is therefore proportional to the stress. After yielding, the growth rate of stress starts decreasing due to the presence of plastic strain. The bottom-right figure in Fig. 3.19 shows the temporal variation of both stress and strain in the bar, as well as the stress–strain relationship.

The following paragraph will cover the creep responses of structures subjected to load-controlled (primary) stress or displacement-controlled (secondary) stress. As an example of load-controlled stress, let's consider a bar with a weight at its end, as shown in Fig. 3.20. In addition, stress and strain vary in time as the bar is heated by a heater after time t_0 with the load kept constant. When the vessel is pressurized at a

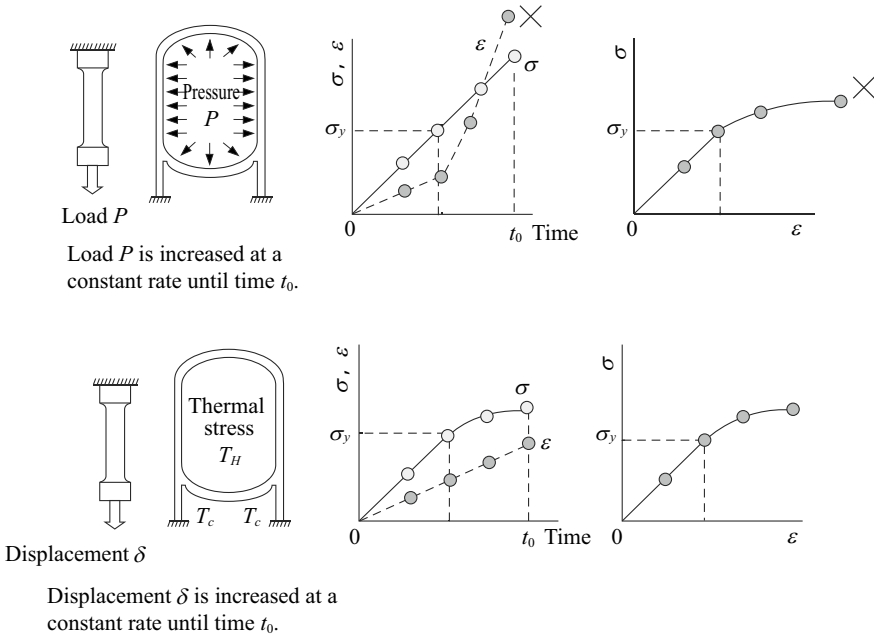


Fig. 3.19 Load-controlled (primary) stress and displacement-controlled (secondary) stress (elastic–plastic deformation)

constant rate and then is heated with the pressure kept constant, the hoop stress and strain in the shell are similar in nature to the stress and strain in the bar.

The stress and strain in the bar are similar to those in Fig. 3.19 until time t_0 . After time t_0 , the load is constant. Therefore, the stress determined by the equilibrium with the external force is also constant. However, the strain increases indefinitely with time due to the creep of the material, which eventually leads to failure. From the above discussion, the temporal variation of the stress and strain in the bar and the stress–strain relationship can be illustrated as shown in the top-right figure in Fig. 3.20. The symbols in the figure correspond with the temporal variation case.

As an example of displacement-controlled stress, let's consider a bar with forced displacement applied at the end as shown in Fig. 3.20, as well as a variation in stress and strain as the bar is heated by a heater after time t_0 with the displacement kept constant. We also consider a vessel heated at a constant rate with the supporting structures fixed on the cold floor and then kept at high temperatures. The stress and strain in the support structures are similar in nature to the stress and strain in the bar, except for the effect of the elastic follow-up, which will be described in the next paragraph. The strain in the bar is determined by geometric compatibility according to the amount of the forced displacement. If the bar is kept at high temperatures after time t_0 , creep strain occurs. However, the total strain is constant, and therefore the

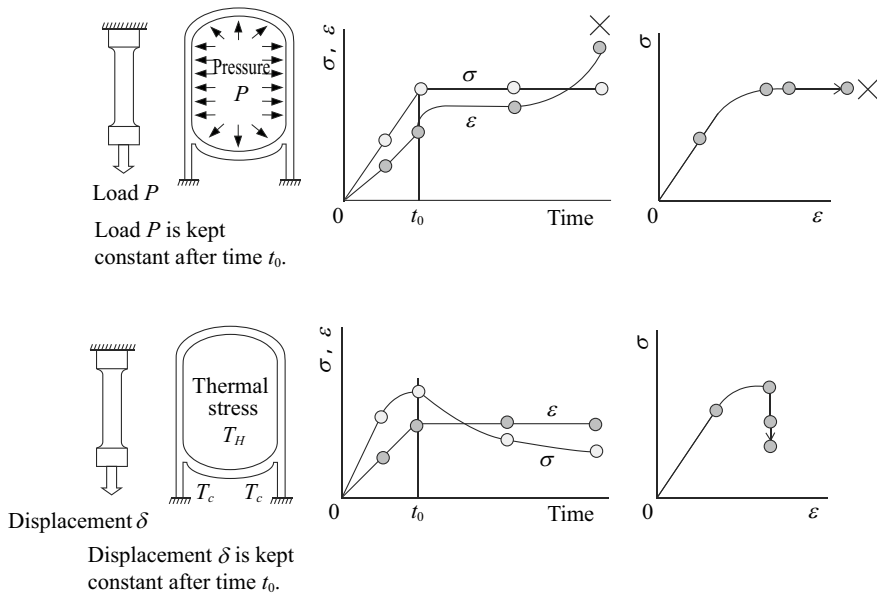


Fig. 3.20 Load-controlled (primary) stress and displacement-controlled (secondary) stress (creep deformation)

elastic strain decreases by the amount of the creep strain, reducing stress accordingly. Temporal variation of stress and strain in the bar, as well as the stress–strain relationship can be illustrated in the bottom-right figure in Fig. 3.20.

3.3.3 Elastic Follow-Up

The stress–strain diagram in Fig. 3.21 compares creep behavior under two types of loadings, load-controlled (primary) stress and displacement-controlled (secondary) stress conditions, in the case where a load is applied to a bar to achieve the same amount of elastic deformation as its initial condition. Under load-controlled conditions, the strain in the bar increases under constant stress. Under displacement-controlled conditions, the load decreases under constant strain. Let's consider a structure (a variable cross-section bar) consisting of two bars connected in series, a high-stiffness/low-stress bar and a low-stiffness/high-stress bar, in which the total elongation of the structure remains constant. The stress–strain behavior of the low-stiffness/high-stress bar is an intermediate behavior between the load- and displacement-controlled loadings.

The above mechanism is described in Fig. 3.21. (1) A high-stiffness/low-stress bar A and a low-stiffness/high-stress bar B are connected in series. (2) Displacement δ is applied at the end of bar B to cause elastic deformation. The relationship

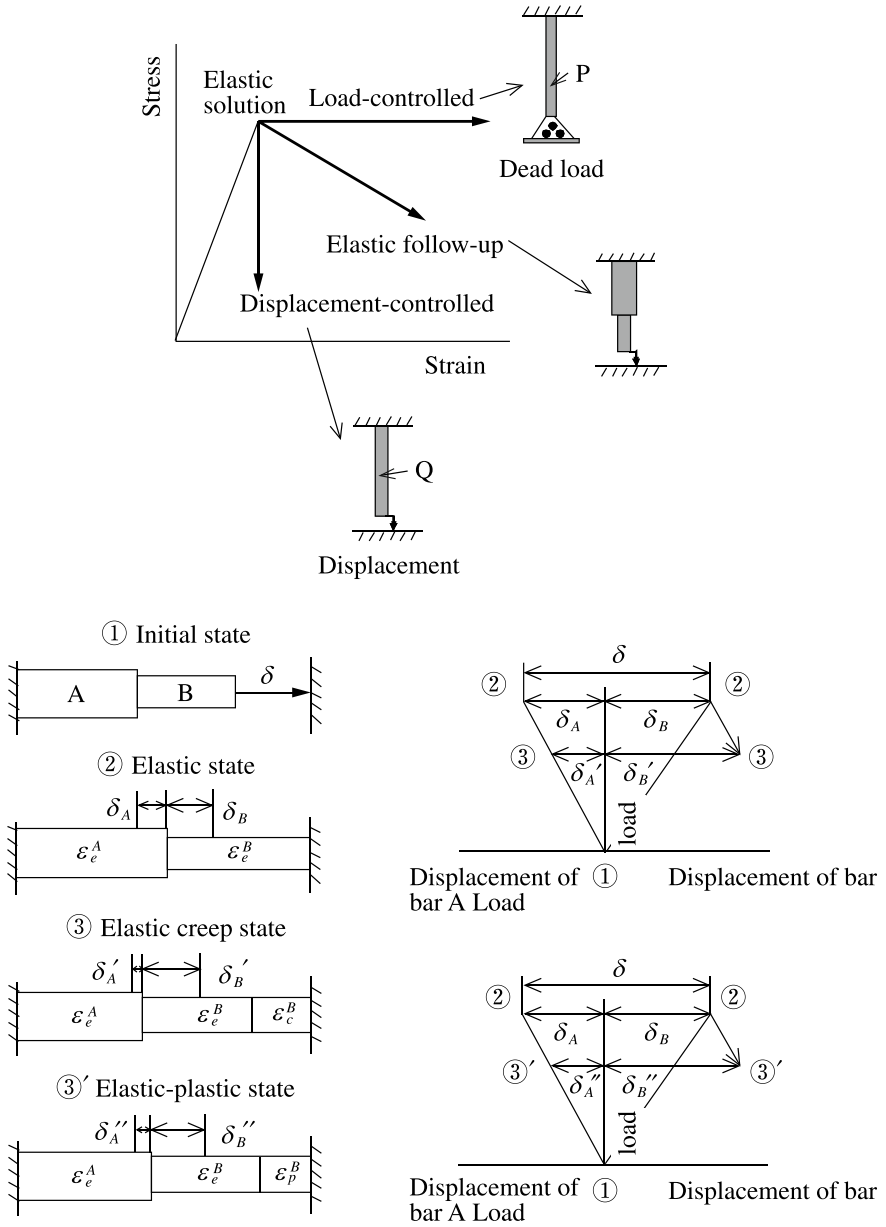


Fig. 3.21 Mechanism of the elastic follow-up

between the displacement of each bar and the load applied equally to both bars is described in a load–displacement diagram. (3) When kept at high temperatures, the creep strain in the variable cross-section bar increases mainly in the low-stiffness/high-stress portion due to the nonlinear stress–strain relationship. Without the high-stiffness/low-stress portion, the elastic strain would be recovered by the amount of the creep strain. In the variable cross-section bar, the stress relaxation occurs in the high-stiffness/low-stress portion due to the creep deformation of the low-stiffness/high-stress portion, and accordingly the elastic recovery of deformation takes place in the high-stiffness/low-stress portion. The increase in strain or deformation in the low-stiffness/high-stress portion (of the amount of the elastic recovery strain) causes excessive strain concentration in this portion, which is not subjected to elastic deformation. (4) A similar phenomenon occurs in elastic–plastic deformation. Without the high-stiffness/low-stress portion, and when plastic strain occurs in the low-stiffness/high-stress portion, the elastic strain would decrease by the amount of the plastic strain, therefore reducing the loading. In the variable cross-section bar, elastic recovery occurs in the high-stiffness/low-stress portion due to this. The strain in the low-stiffness/high-stress portion increases by the amount of this elastic recovery strain.

This phenomenon is called elastic follow-up because the low-stiffness/high-stress portion behaves as if it were followed by the elastic recovery of the high-stiffness/low-stress portion.

Components and structures have both high-stiffness/low-stress and low-stiffness/high-stress portions, and are therefore rarely subject to complete load- or displacement-controlled loading. Usually, some amounts of elastic follow-up would occur. The problem is that even if the load to the entire system is displacement-controlled, the strain in the low-stiffness/high-stress portion is not self-controlled. Moreover, stresses that are considered as displacement-controlled stress in stress classifications can often behave in a load-controlled manner. In other words, even though self-controlled deformation is expected, actual deformation may become larger than expected, resulting in unexpected structural failure.

3.3.4 Stress Concentration

Some reactor components and structures have abrupt changes in cross section, and stress concentration may occur near such locations. Stress concentration is described using, as an example, a flat plate with a hole subjected to a tensile load in Fig. 3.22. The distribution of the tensile stress in the section through the center of the hole and along the normal section to the load shows that the maximum tensile stress occurs at the edge of the hole and decreases drastically away from the edge. This phenomenon is called stress concentration and occurs because the edge of the hole must support the force that would have been supported by the plate section replaced by the hole.

The average stress in the cross section is called membrane stress σ_m and the maximum stress at the edge is called peak stress σ_p . These stresses are distinguished

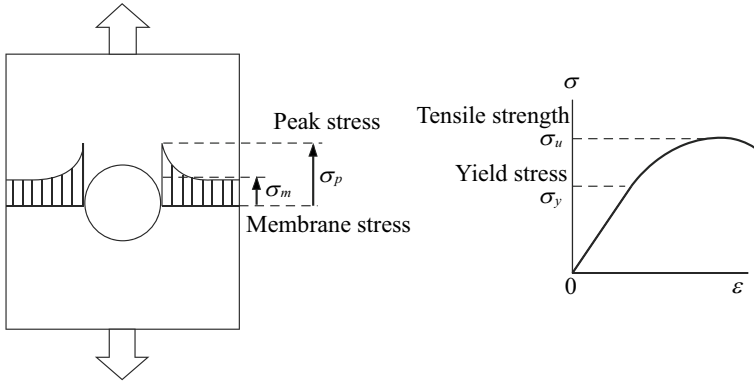


Fig. 3.22 Stress concentration (membrane stress and peak stress)

because they affect structural failure differently.

$$\sigma_m < \sigma_p < \sigma_y \quad (3.31)$$

In this case, the entire structure is in the elastic region. If the load increases and we have

$$\sigma_m < \sigma_y \leq \sigma_p \quad (3.32)$$

Then, the plate yields locally at the edge of the hole. Even in this case, most of the cross section is in the elastic state and therefore, excessive deformation or fracture of the entire structure does not occur. If the load is repeated, cracking induced by low-cycle fatigue could occur at the edge of the hole subjected to plastic cycles. Consequently, peak stresses affect material fatigue but do not contribute to the global deformation and ductile fracture of the entire structure.

3.4 Failure Modes of Structures

3.4.1 Ductile Fracture, Plastic Collapse, and Creep Rupture

In this section, you will learn about typical failure modes assumed in the design codes and standards for reactor components and structures²⁾.

We assume that in Fig. 3.22, the load further increases from the state in Eq. (3.32) and we have

$$\sigma_m < \sigma_y \leq \sigma_p \quad (3.33)$$

then, the entire plate yields and excessive plastic deformation occurs. If we have

$$\sigma_u \leq \sigma_m \quad (3.34)$$

then, ductile fracture occurs in the cross section of the plate through the center of the hole.

Let $\sigma(t)$ be the creep rupture stress determined by the operating temperature and time t . Then, for high temperatures, we have

$$\sigma(t) \leq \sigma_m \quad (3.35)$$

Creep rupture occurs in the cross section through the center of the hole within time t .

Let $t_R(\sigma_m)$ be the creep rupture time determined by the operating temperature and the membrane stress σ_m . Creep rupture occurs for the operation time t

$$t_R(\sigma_m) \leq t \quad (3.36)$$

These results show that the membrane stress is a stress component which can directly cause rupture and deformation.

Furthermore, bending stress σ_b is discussed using an elastic-perfectly plastic beam subjected to a bending moment as shown in Fig. 3.23. The strain in the beam is linearly distributed across the section of the beam, as illustrated in Fig. 3.23a, derived from the compatibility conditions applied to the plane. In an elastic beam, the stress is proportional to the strain. In an elastic-perfectly plastic beam, yielding occurs at the top and bottom surfaces, and furthermore, yield stresses with opposite signs are distributed around the neutral axis across the section of the beam. For the height of the elastic region $\pm c/2$, the bending moment is given by

$$M(c) = \int \sigma_y dA = \int_{-c/2}^{c/2} E \varepsilon_y b dy + 2 \int_{c/2}^{h/2} \sigma_y y b dy = \left(\frac{bh^2}{4} \right) \sigma_y \left\{ 1 - \frac{1}{3} \left(\frac{c}{h} \right)^2 \right\} \quad (3.37)$$

The bending moment corresponding to the case where the stress on the surface is at yield point, as well as the bending moment corresponding to the case where the beam yields across its entire cross section, respectively given by

$$M(h) = \frac{2}{3} \left(\frac{bh^2}{4} \right) \sigma_y, \quad M(0) = \left(\frac{bh^2}{4} \right) \sigma_y \quad (3.38)$$

The beam has a maximum loading capacity of $M(0)$ and can support a bending moment 1.5 times as large as its bending moment $M(h)$, which corresponds to surface

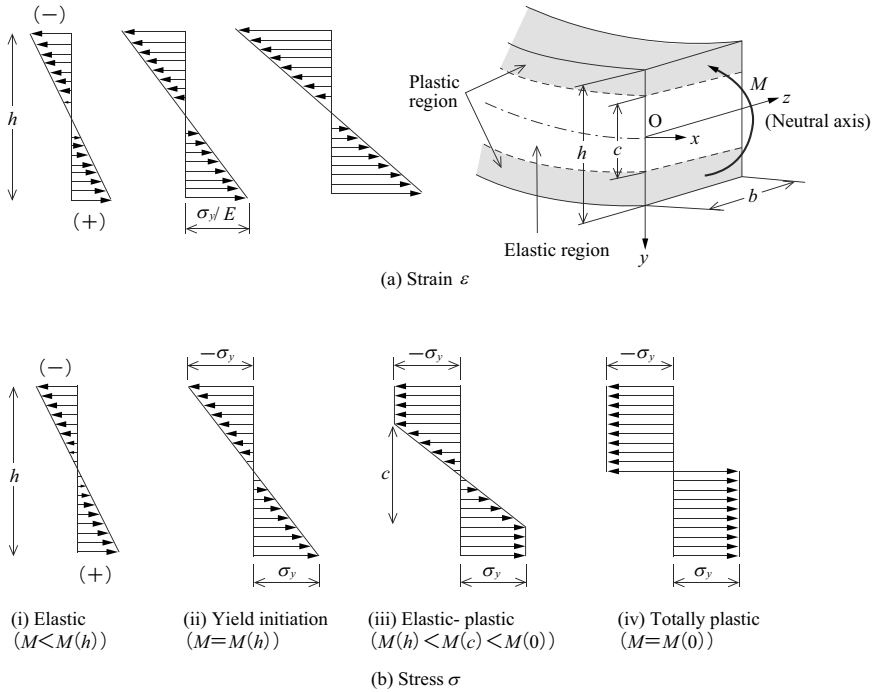


Fig. 3.23 An elastic-perfectly plastic beam subjected to bending moment

stress reaching the yield stress. This state is called plastic collapse and is treated as one of the failure modes.

3.4.2 Deformation Due to Cyclic Loading

When cyclic displacement-controlled stress is superimposed on constant load-controlled stress and the combination of these stresses exceeds a certain limit, deformation gradually increases and accumulates in the direction of the load-controlled stress applied, as the number of cycles of the displacement-controlled stress increases. This phenomenon is called ratchet deformation. For example, let's consider two cases, displayed in Fig. 3.24: a weight is hung on a tube to apply load-controlled stress, and a tube in which hot and cold fluids are passed alternately through it, to apply cyclic displacement-controlled stress. In the latter case, even when stress exceeding the yield stress is applied, the stress-strain cycles become stable as the number of cycles increases and eventually takes the same pattern (plastic cycles). Even if the loads that are superimposed that cannot increase the deformation when applied individually, still the tube will gradually extend in the direction of the tensile

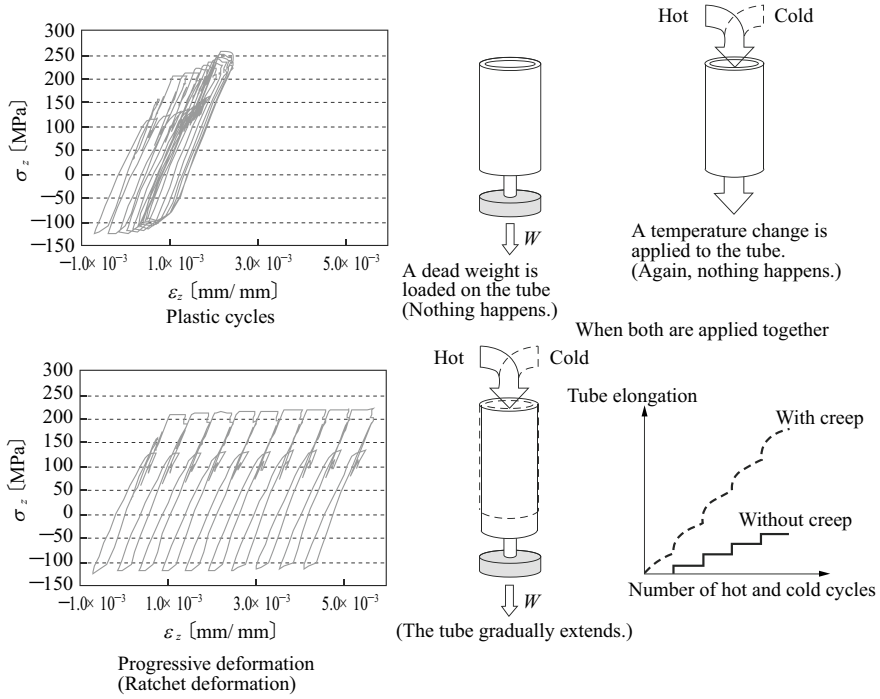


Fig. 3.24 Plastic cycles and ratchet deformation

load, which is given by the weight as the number of thermal cycles increases. Such progressive deformation caused by cyclic thermal stress is called thermal ratchet.

The basic mechanism of the thermal ratchet is described using a two-bar model in Fig. 3.25. The two bars are fixed at the top and connected to a horizontally kept rigid body at the bottom. They are subjected to a constant tensile load via the rigid body. They are assumed to be elastic-perfectly plastic, and have the same material, shape, and initial temperature.

As shown in the figure, bar 2 is subjected to cycles of heating and cooling with bar 1 kept at a constant temperature. Then, the steady stress in bars 1 and 2 is given by

$$\sigma_1 = \sigma_2 = \sigma_m = P/(2A) \quad (3.39)$$

where σ_m is the constant membrane stress, P is the constant tensile load, and A is the cross-sectional area of each bar.

Let ΔT ($T_2 \geq T_1$) be the maximum temperature difference between the two bars. Then, the thermal stress is given by

$$\sigma_{t1} = -\sigma_{t2} = \sigma_t = \frac{1}{2}E\alpha\Delta T \quad (3.40)$$

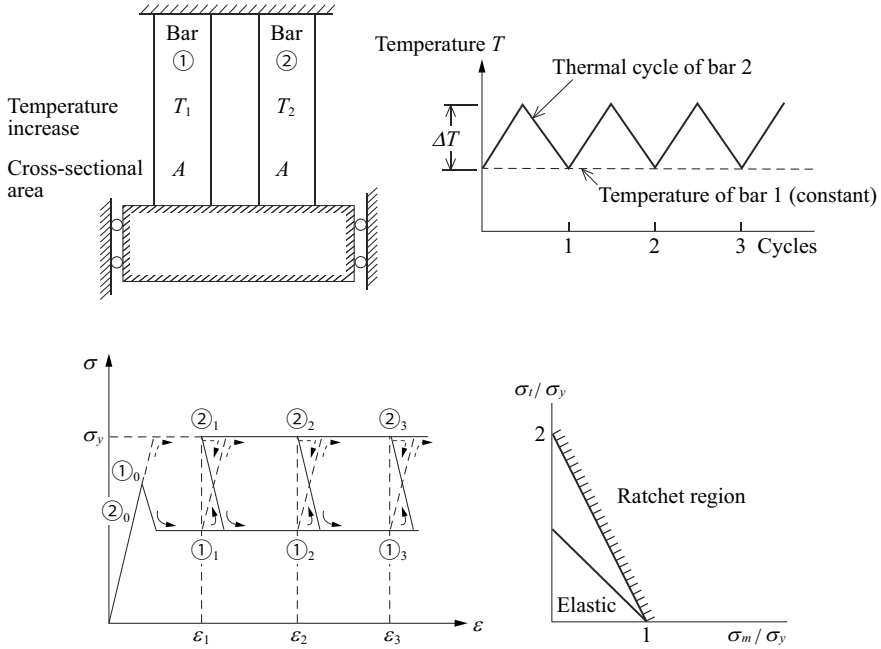


Fig. 3.25 Basic mechanism of thermal ratchet

where σ_t is the maximum thermal stress.

We consider the superposition of the steady-state membrane stress σ_m and the thermal stress σ_t . The conditions for bar 1 to remain in the elastic region are

$$\sigma_m + \sigma_t \leq \sigma_y \quad (3.41)$$

When the stress in bar 1 exceeds the yield stress and enters the plastic region, stress-strain hysteresis loops in Fig. 3.25 are formed, and the accumulation of strain (ratchet) occurs at every cycle while plastic strain is generated alternately in the tensile direction. The amount of the accumulated strain per cycle is given by

$$\Delta \varepsilon = 4(\sigma_m - \sigma_y)/E + \alpha \Delta T \quad (3.42)$$

The conditions for ratchet to occur due to thermal stress are

$$\sigma_m + \frac{1}{2} \sigma_t \leq \sigma_y \quad (3.43)$$

This equation shows that the ratchet region of the two bars can be represented by a combination of load- and displacement-controlled stresses as shown in Fig. 3.25.

3.4.3 *Initiation and Propagation of Fatigue and Creep-Fatigue Cracks*

As described in Sect. 3.3.4, structures have regions of stress concentration due to structural discontinuities. Even if a structure is designed to remain in the elastic range, plastic cycles are applied locally due to cyclic loading and cracks may occur due to fatigue (especially creep-fatigue at high temperatures) in the area where the plastic cycles are applied. If the cyclic loading continues, the cracks may propagate and penetrate through the thickness of the structure. Figure 3.26 shows an example of cracks due to cyclic thermal stress observed in a structure test. The figure shows that the cracks occurred in an area of stress concentration, caused by the geometric discontinuity and at the edge of the weld joint.

Figure 3.27 shows the results of an inelastic finite element simulation which calculates the redistribution of stress and strain in the root of the Y-shaped support welded to the main shell where thermal stress has been generated by creating a temperature difference between the support and the shell, kept at high temperatures. The figure indicates that the creep region spreads with time and the increase of strain progresses, while stress is reduced. If the loading is applied cyclically, the strain concentration becomes more significant, leading to a reduced operational lifetime for the structure.

Fig. 3.26 Cracks caused by cyclic thermal stress

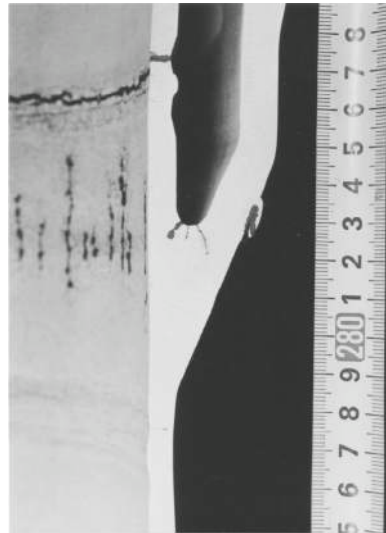
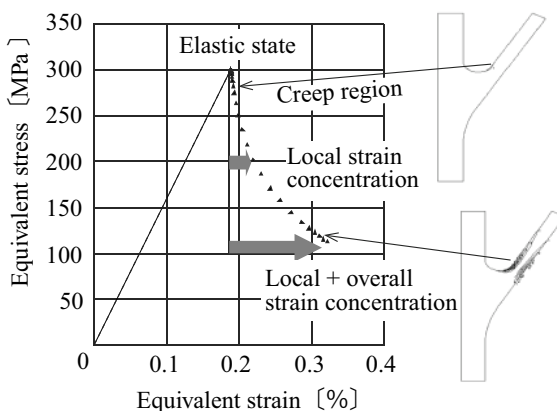


Fig. 3.27 Strain concentration due to creep at structural discontinuities



3.4.4 Buckling

In Chap. 2, we learned about Euler's buckling phenomenon observed in a slender column subjected to compressive load. The column contracts in a straight manner when the axial compressive load is below a certain value (limit), but deflects suddenly in the lateral direction when the load exceeds the limit. Buckling might occur in reactor structures such as vessels and pipes when structures are subjected to external pressure, axial compressive loads, or bending loads. When an axial compressive load is applied to a structural member, different buckling modes can occur depending on the type of the load and the constraint conditions. Some buckling modes involve a significant decrease in buckling load due to manufacturing imperfections of shape in structures. Post-buckling behavior significantly varies with the buckling mode, ranging from gradual deformation to sudden failure. Due to the importance of understanding the buckling modes of structures, the following classifications are used:

(1) Classification focusing on the stress–strain relationship of a material

- Elastic buckling or elastic instability: Buckling in the range of the elastic stress–strain relationship of a structure or its members. Parameters involved in elastic buckling are elastic constants such as Young's modulus and structural geometric factors such as the aspect ratio of columns. They are not involved in the strength of a material, such as the yield stress.
- Plastic buckling or plastic instability: Buckling beyond the elastic range of the stress–strain relationship

(2) Classification focusing on the characteristics of loads

- Load-controlled buckling: Buckling leading to failure due to further application of load in post-buckling regime, or buckling caused by load-controlled

stress. A typical example is the collapse of a pipe subjected to external pressure.

- Displacement-controlled buckling: Such buckling involves the reduction of the load, due to a decrease of the strain and the buckling. When the deformation after buckling finds a strain equilibrium, the load disappears. This type of buckling is considered to be a failure mode from the viewpoint of fatigue prevention, as well as excessive deformation from buckling. A typical example is a case in which thermal expansion is constrained.

As shown by these definitions, buckling can be caused by both load- and deformation-controlled stresses.

(3) Classification focusing on deformation modes

- Shear buckling and bending buckling: There are many modes of deformation by buckling. One of them is described below. When a plate is subjected to a bending moment, tensile stress is generated on one side of the plate and compressive stress is generated on the other side. Then, buckling could occur on the compressive stress side. This is called bending buckling. On the other hand, in a pure shear stress state, compressive and tension stresses occur in the $\pm 45^\circ$ directions, causing buckling on the compressive stress side. This is called shear buckling.

Figure 3.28 illustrates an example of shear buckling. It shows the relationship between a load Q and lateral displacement δ when thin cylinders of stainless steel, fixed at the bottom, with same ratio R/t of radius R to thickness t and different ratios L/R of length L to radius R , are subjected to lateral displacement at their top. The short cylinder with $L/R = 1.0$ can tolerate a large load by elastic deformation. Once it buckles, the load decreases suddenly and wrinkles in the shear direction appear over the surface of the cylinder (shear buckling). This type of buckling is so dangerous that adequate design margins are required against buckling. The slender cylinder with $L/R = 4.0$ has a low buckling load, but the decrease in post-buckling load is gradual. Bending occurs due to local plastic deformation in the cylinder (bending buckling). Usually, the behavior of plastic buckling is mild compared to elastic buckling, and safety margins can therefore be reduced. For $L/R = 2.0$, both deformation modes occur simultaneously. As mentioned above, the buckling load and the deformation mode depend on the geometric shape of the structure and the type of loads applied to it.

Exercises for This Chapter

1. The three figures below show the basic characteristics of deformation and strength of a material. What characteristic does each figure show? Write the terms corresponding to numbers 1–8 in the figures. Write the number first, followed by the term.

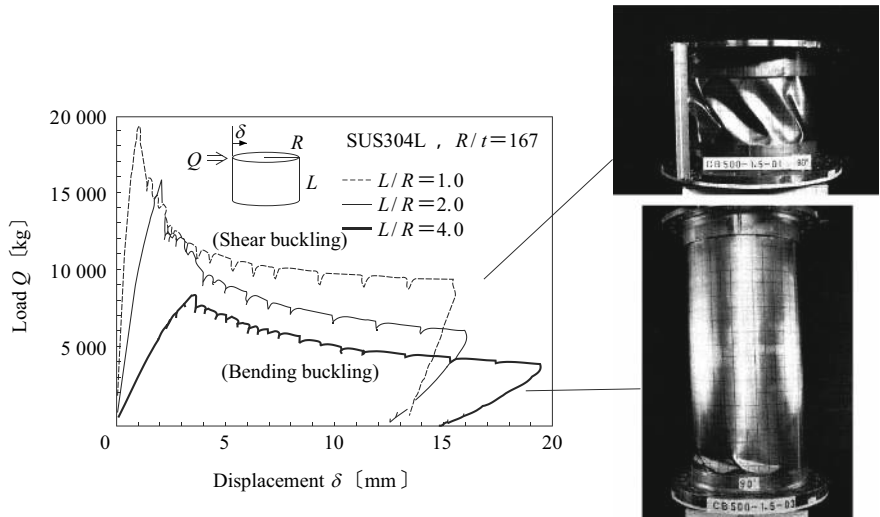
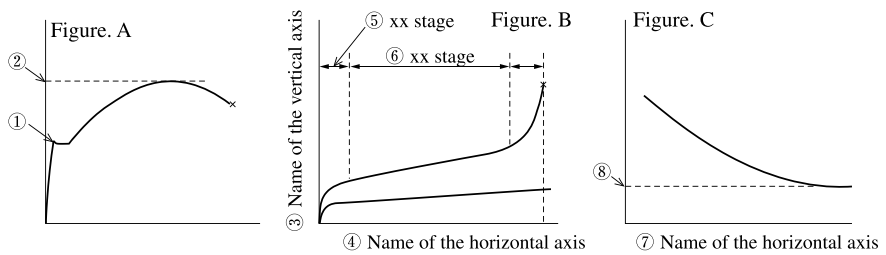


Fig. 3.28 Buckling behavior of thin cylindrical shells subjected to a lateral load



Basic characteristics of deformation, strength, and toughness of a material

2. Choose the correct terms for the blanks in the following text from the list provided at the end of the text. Write the alphabetical letter first, followed by the number of the term.

When a load is applied to a ductile material and kept constant, (A) increases with time. This phenomenon is called (B). It is not a problem at room temperature but important for structures used at high temperatures such as in the reactor. When (C) is kept constant, the stress decreases with time. This phenomenon is called (D). These phenomena are noticeable in metals at high temperatures and in low-melting point metals at room temperature. The relationship between (E) and time is as shown in the figure. This curve is called (F) and is divided into three stages I, II, and III. In stage I, (G) increases with time and is called (I) or (J) in the range in which its increase rate decreases. Stage II is the stage in which work hardening and its recovery are balanced and called (K) or (L). Stage III is the stage in which (M) rapidly increases due to small cracks caused by the combination of voids accumulated in the grain boundary, and is called (N) or (O).

The increase rate of strain with time is called (P). The increase rate in stage II is called (Q). Generally, if the stress is such that (R) is zero in stage II, (S) stops after a certain period of time. The maximum value of this stress is called (T).

(1) elasticity (2) plasticity (3) strain (4) stress (5) ductility (6) brittleness (7) crack (8) fracture.

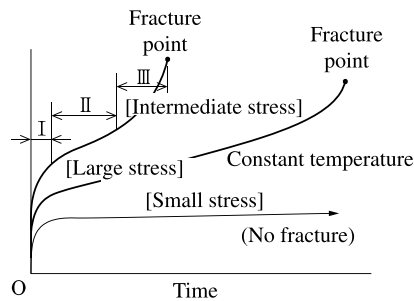
(9) fatigue (10) creep (11) primary creep (12) secondary creep (13) tertiary creep (14) transient creep.

(15) steady-state creep (16) accelerated creep (17) creep rate (18) minimum creep rate.

(19) maximum creep rate (20) creep limit (21) stress–strain curve (22) creep curve.

(23) stress relaxation (24) work hardening (25) softening.

(Terms can be used more than once.)



Temporal change in the properties of a high-temperature material

3. Describe the characteristics of the load applied to the following structures and the failure mode due to the load to which you should pay the highest attention. justify your answers

- (1) A 304SS high-pressure vessel kept at a constant operating temperature of 300 °C
- (2) A 304SS high-pressure vessel kept at a constant operating temperature of 500 °C
- (3) A 304SS low-pressure vessel subjected to cyclic temperature changes between room temperature and 300°C
- (4) A 304SS high-pressure vessel subjected to cyclic temperature changes between room temperature and 300°C

Answers to Exercises

1. (1) yield stress, (2) tensile strength, (3) creep strain, (4) time, (5) primary (transient) creep, (6) secondary (steady) creep, (7) failure cycle number, (8) endurance limit
2. A: (3) strain, B: (10) creep, C: (3) strain, D: (23) stress relaxation, E: (3) strain, F: (22) creep curve, G: (3) strain, H: (24) work hardening, I: (11) primary creep,

- J: (14) transient creep, K: (12) secondary creep, L: (15) steady creep, M: (3) strain, N: (13) tertiary creep, O: (16) accelerated creep, P: (17) creep rate, Q: (18) minimum creep rate, R: (17) creep rate, S: (10) creep, T: (20) creep limit
3. (omitted).

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Chapter 4

Structural Analysis by the Finite Element Method



Abstract Stress and strain generated in structural components with complicated shapes and boundary conditions are often impossible to solve with only theoretical methods from structural mechanics and strength of materials. With rapid progress in computers and computational science, numerical analysis methods have been increasingly applied to such problems. Among various numerical analysis methods, the finite element method is widely used as it is highly applicable to structural analysis of complicated shapes and can accurately predict the local distribution of stress and strain.

Stress and strain generated in structural components with complicated shapes and boundary conditions are often impossible to solve with only theoretical methods from structural mechanics and strength of materials. With rapid progress in computers and computational science, numerical analysis methods have been increasingly applied to such problems. Among various numerical analysis methods, the finite element method is widely used as it is highly applicable to structural analysis of complicated shapes and can accurately predict the local distribution of stress and strain. Many commercial computer programs based on the finite element method are available and widely used for analysis of reactor structures. In order to use these programs to analyze reactor structures that require high reliability, it is crucial to develop proper models to verify the solution obtained. This chapter briefly describes the theory of the finite element method, as well as a method for developing a model and verifying the solution. Knowledge of strength of materials is required to understand and use the finite element method. It is recommended to study this chapter after proper understanding of the previous chapters.

4.1 Theory of the Finite Element Method

4.1.1 Concept of the Finite Element Method

The applicability of theoretical solutions to structures with complicated shapes is limited. Therefore, numerical calculations are widely used for analyses of reactor components. Local geometries must be considered to accurately predict stresses. Among various numerical calculation methods, the finite element method (FEM) is one of the most used, as it is applicable to any shape. There are many detailed books on the FEM, as refereed at the end of this chapter. Use of commercial FEM software has become quite common as well. Considering these facts this chapter will provide an overall picture of the FEM, and will cover the knowledge necessary to develop models using currently available programs and verify the solution obtained.

FEM is a numerical analysis technique which derives deformation, stress, and strain by representing a complex structure by a set of segments called elements, with specific degrees of freedom (Fig. 4.1), and solving the matrix-form of equilibrium equations in the set of elements. Adjacent elements are connected by their nodal points (or nodes), which are included in their common boundary. The entire structure has to be represented by a set of elements. Necessary physical quantities associated with the object of the analysis, such as displacement and nodal forces are assigned to the node. The equilibrium equation for the whole set of elements is obtained by assembling the equilibrium equations for all elements. Three methods are available to formulate the equilibrium equations for finite elements:

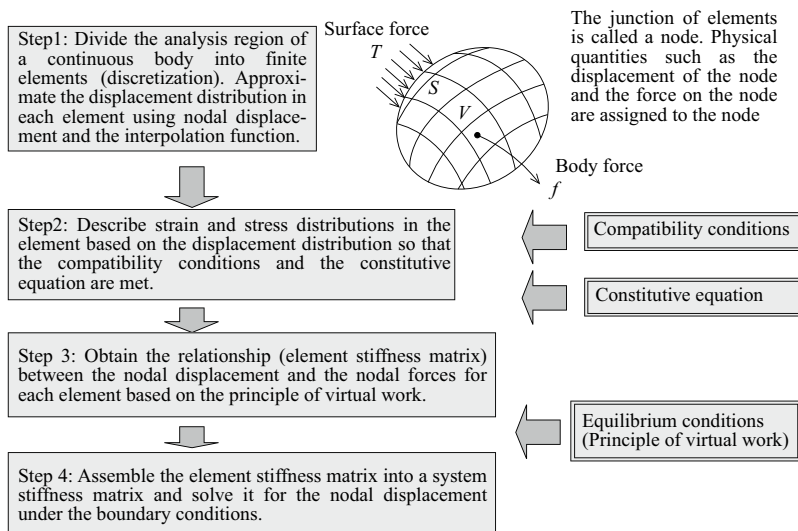


Fig. 4.1 Outline of structural analysis by FEM (using the displacement method)

- The displacement method, which defines displacement components as unknown degrees of freedom;
- The stress method, which defines stress as an unknown degree of freedom;
- The hybrid method, which combines the two previous methods.

In practice, the displacement method, which offers ease of use and consistency of computation, is the dominant method [1].

The overall picture of the FEM concept is described in Fig. 4.1 through four steps, focusing on the displacement method. This chapter will only cover elastic analysis, as thermal analysis and inelastic analysis will be discussed in Chap. 6.

[Step 1] Generate a mesh and set a displacement function

Divide the analysis region of a continuous body into finite elements as shown in Fig. 4.1. Displacement and reaction force are assigned to the nodes of each element. Then, select a function $N_{(k)}$ (mostly a polynomial) called a displacement function for each element (for example, element k). Approximate by interpolation the displacement distribution in the element from the nodal displacement using the function.

$$\{u_{(k)}\} = [N_{(k)}]\{U_{(k)}\}, \quad \text{on } V^{(k)} \quad (4.1)$$

where $\{u_{(k)}\}$ is the displacement in element (k), $[N_{(k)}]$ is the displacement function for element (k), and $\{U_{(k)}\}$ is the nodal displacement of element (k).

[Step 2] Introduce Compatibility Conditions and Constitutive Equations

As covered in Chap. 2, the basic equations in structural mechanics are “equilibrium conditions,” “compatibility conditions,” and “constitutive equations.” From Eq. (4.1), obtain strain–displacement and stress–strain relationships for the element so that the compatibility conditions and the constitutive equations are met.

Normally, the strain in the element is determined from the geometric conditions as a function of the derivative of displacement, as follows:

Compatibility conditions (strain–displacement relationship)

$$\{\varepsilon\} = [\partial]\{u\}, \quad \delta\{\varepsilon\} = [\partial]\{\delta u\} \quad (4.2)$$

The stress is obtained from the strain using the constitutive equation (stress–strain relationship) for the material, as follows. For an elastic body, the constitutive equation is given in terms of stress–strain matrix $[D]$ [Eq. (3.2)] and the initial strain ε^0 , as follows:

$$\text{Constitutive equation (stress - strain relationship)} \{\sigma\} = [D](\{\varepsilon\} - \{\varepsilon^0\}) \quad (4.3)$$

[Step 3] Derive an Element Stiffness Equation that Meets the Equilibrium Conditions.

We can derive the equilibrium conditions for a finite element from the principle of virtual work or the principle of minimum potential energy. How to derive the element stiffness equation between the nodal displacement and force for element (k) was previously described in Chap. 2, using the principle of virtual work.

The equilibrium conditions (the principle of virtual work) for element (k) are given by

$$\int_{V^{(k)}} \{\sigma\}^T \delta\{\varepsilon\} dV = \int_{V^{(k)}} \{f\}^T \delta\{u\} dV + \int_{S^{(k)}} \{T\}^T \delta\{u\} dS \quad (4.4)$$

Strain energy Work by body force Work by surface force.

Substituting Eqs. (4.1)–(4.3) into Eq. (4.4), we obtain the stiffness equations for element (k).

$$[K_{(k)}] \{U_{(k)}\} = \{F_{(k)}\} \quad (4.5)$$

$$[K_{(k)}] = \int_{V^{(k)}} [B]^T [D] [B] dV \quad (4.6)$$

$$[B] = [\partial] [N] \quad (4.7)$$

$$\{F_{(k)}\} = \int_{V^{(k)}} [N]^T \{f\} dV + \int_{S^{(k)}} [N]^T \{T\} ds + \int_{V^{(k)}} [B]^T [D] \{\varepsilon^0\} dV \quad (4.8)$$

$[K_{(k)}]$ in Eq. (4.6) is called the element stiffness matrix and $\{F_{(k)}\}$ in Eq. (4.8) is called the element load vector. For complex structures, it is difficult to solve analytically the equation which meets the equilibrium conditions. Therefore, the principle of virtual work which has been demonstrated to be equivalent to the equation of equilibrium (see Sect. 2.9), is used instead.

[Step 4] Obtain and Solve the Global Stiffness Equation

Nodes with same coordinates are identical nodes, thus the displacement of these nodes is identical, and the nodal forces are balanced. Taking this into consideration, we assemble the element stiffness Eq. (4.5) for all elements to obtain a global system of stiffness equations.

$$[K] \{U\} = \{F\} \quad (4.9)$$

In Eq. (4.9), $[K]$ is called the system stiffness matrix. It is a square matrix having a number of elements per row or column equal to the number of nodes times the number of degrees of freedom (3 in a three-dimensional problem). The global system

of stiffness equations represents the equilibrium conditions. Displacements and reaction forces are obtained for all nodes by solving this equation under the boundary conditions. The strain in the element can be obtained by substituting the displacement into Eq. (4.2) via Eq. (4.1). Then, the stress can be obtained by using Eq. (4.3).

[Supplement: The Principle of Virtual Work and the Principle of Minimum Potential Energy]

It has been previously demonstrated that the principle of virtual work, which describes the equilibrium conditions in the displacement-based FEM, is equivalent to Castigliano's theorem (Sect. 2.5) and to the principle of minimum potential energy (Sect. 2.9).

The principle of virtual work in Eq. (4.4) can be simplified to the following equation in terms of strain energy U , an external force P , and displacement u :

$$\delta U = P\delta u \quad (4.10)$$

This equation means that when a small virtual displacement is applied to an elastic body in equilibrium, the sum $P\delta u$ of the work by the forces is equal to the strain energy δU .

As a result, Castigliano's theorem mentioned in Sect. 2.5 in Chap. 2 is derived as

$$\frac{dU}{du} = P \quad (4.11)$$

The potential energy of the system Π is given by

$$\Pi = U - Pu \quad (4.12)$$

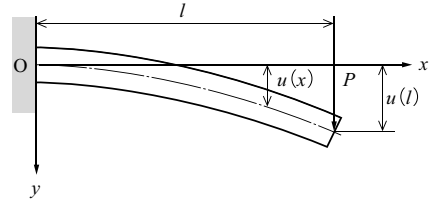
From Eq. (4.10), we obtain

$$\delta \Pi = \delta U - P\delta u = 0 \quad (4.13)$$

Equation (4.13) represents the principle of minimum potential energy stating that the potential energy of an elastic body reaches a minimum in an equilibrium state.

Next, let's describe the Rayleigh–Ritz method, which is an approximation technique based on the principle of minimum potential energy. It provides the basis to conduct the FEM. This method assumes, based on the above principle, that when displacement (or stress) meets the boundary conditions, the solution exists and is a trial function in which the displacement is a linear combination of Eigen functions multiplied by unknown constants. These unknown constants are determined when a minimum is reached for the potential energy. An application of this method would be, for example, the deflection of a cantilever beam with a length l , subjected to a concentrated load P as illustrated below in Fig. 4.2.

Fig. 4.2 A cantilever beam subjected to a concentrated load P at its end



Deflection u is approximated by a polynomial of x so that the boundary conditions for the fixed end, $u(0) = 0$ and $[du(x)/dx]_{x=0} = 0$, are met. Here, $a_2, a_3, a_4 \dots$ are unknown constants.

$$u(x) = a_2x^2 + a_3x^3 + a_4x^4 + \dots \quad (4.14)$$

The bending moment is given by

$$M = -EI \frac{d^2u}{dx^2} = -EI(2a_2 + 6a_3x + 12a_4x^2 + \dots) \quad (4.15)$$

The strain energy in terms of bending moment is given by

$$U = \frac{1}{2EI} \int_0^l M^2 dx \quad (4.16)$$

The virtual work done by the load when the displacement of the point of load application changes by $u(l)$ under the constant load P is given by

$$Pu(l) = P(a_2l^2 + a_3l^3 + a_4l^4 + \dots) \quad (4.17)$$

The unknown constants $a_2, a_3, a_4 \dots$ are determined to meet the following stationary conditions by substituting Eqs. (4.16) and (4.17) into Eq. (4.13) and using the principle of minimum potential energy.

$$\frac{\partial\{U - Pu(l)\}}{\partial a_2} = 0, \quad \frac{\partial\{U - Pu(l)\}}{\partial a_3} = 0, \quad \frac{\partial\{U - Pu(l)\}}{\partial a_4} = 0 \quad (4.18)$$

The following equation, which calculates the deflection of the beam, is obtained by substituting the previously derived unknown constants into Eq. (4.14):

$$u(x) = \frac{Pl^3}{6EI} \cdot \frac{x^2}{l^2} \left(3 - \frac{x}{l}\right) \quad (4.19)$$

4.1.2 Examples of Application of the FEM (a One-Dimensional Problem)

In this paragraph, the FEM is applied specifically to a one-dimensional problem, by following the four steps concept described previously. Let's consider the problem of bar with a variable section, under elastic tension (Fig. 4.3 [2]). The bar is divided into elements A and B , with respective Young's modulus E_i ($i = A, B$) and cross-sectional areas S_i ($i = A, B$). As for the boundary conditions, the displacement at the left end of the bar ($x = x_1$) is $u_1 = 0$ and a load X_3 is applied at the right end of the beam ($x = x_3$).

[Step 1] Set a Displacement Function for a One-Dimensional Element.

We represent the element displacement in elements A and B with nodal displacement. First, let's consider element A . We assume that the displacement u in the x -direction is given by a linear function of x . Then, α_1 and α_2 are given by the following equation as constants independent from the coordinate x :

$$u = \alpha_1 + \alpha_2 x \quad (4.20)$$

Let u_1 and u_2 be the displacement at the left ($x = x_1$) and right ($x = x_2$) ends of element A . Then, the above equation is rewritten as

$$u_1 = \alpha_1 + \alpha_2 x_1, \quad u_2 = \alpha_1 + \alpha_2 x_2 \quad (4.21)$$

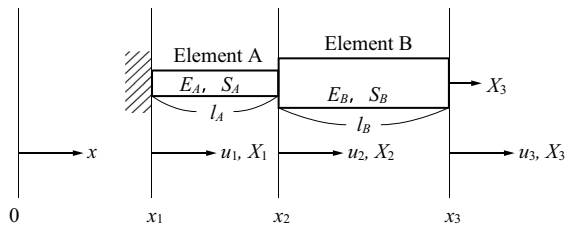
Solving this equation for $\{\alpha_1, \alpha_2\}$, we obtain

$$\{\alpha\} \equiv \begin{Bmatrix} \alpha_1 \\ \alpha_2 \end{Bmatrix} = \begin{bmatrix} 1 & x_1 \\ 1 & x_2 \end{bmatrix}^{-1} \begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} = \frac{1}{l_A} \begin{bmatrix} x_2 & -x_1 \\ -1 & 1 \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} \quad (4.22)$$

Substituting Eq. (4.22) into Eq. (4.20), we obtain

$$u = \begin{bmatrix} 1 - \frac{x-x_1}{l_A} & \frac{x-x_1}{l_A} \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} \equiv [N_A] \begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} \quad (4.23)$$

Fig. 4.3 The elastic variable-section bar model subjected to tensile load



Similarly, for element B , we obtain

$$u = \left[1 - \frac{x-x_2}{l_B} \quad \frac{x-x_2}{l_B} \right] \begin{Bmatrix} u_2 \\ u_3 \end{Bmatrix} \equiv [N_B] \begin{Bmatrix} u_2 \\ u_3 \end{Bmatrix} \quad (4.24)$$

$[N_A]$ and $[N_B]$ in Eqs. (4.23) and (4.24) are matrices relating the element displacement u to the nodal displacement $\{u\}$, and are called the interpolation or shape functions.

[Step2] Introduce Compatibility Conditions and a Constitutive Equation into a One-Dimensional Element

First, from Eq. (4.23), the strain ε_x^A and strain variation $\delta\varepsilon_x^A$ in element A which meet the compatibility conditions from the geometric relationship are given by

$$\varepsilon_x^A = \frac{du}{dx} = \left[-\frac{1}{l_A} \quad \frac{1}{l_A} \right] \begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix}, \quad \delta\varepsilon_x^A = \frac{d\delta u}{dx} = \left[-\frac{1}{l_A} \quad \frac{1}{l_A} \right] \begin{Bmatrix} \delta u_1 \\ \delta u_2 \end{Bmatrix} \quad (4.25)$$

The constitutive equation for a one-dimensional elastic body is given by Hooke's law. Then, the stress σ_x^A in element A is given by the following equation in terms of ε_x^A :

$$\sigma_x^A = E_A \varepsilon_x^A = E_A \left[-\frac{1}{l_A} \quad \frac{1}{l_A} \right] \begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} \quad (4.26)$$

Similarly, from Eq. (4.24), the strain, the strain variation, and the stress in element B are given by

$$\varepsilon_x^B = \frac{du}{dx} = \left[-\frac{1}{l_B} \quad \frac{1}{l_B} \right] \begin{Bmatrix} u_2 \\ u_3 \end{Bmatrix}, \quad \delta\varepsilon_x^B = \frac{d\delta u}{dx} = \left[-\frac{1}{l_B} \quad \frac{1}{l_B} \right] \begin{Bmatrix} \delta u_2 \\ \delta u_3 \end{Bmatrix} \quad (4.25')$$

$$\sigma_x^B = E_B \varepsilon_x^B = E_B \left[-\frac{1}{l_B} \quad \frac{1}{l_B} \right] \begin{Bmatrix} u_2 \\ u_3 \end{Bmatrix} \quad (4.26')$$

[Step 3] Derive the Stiffness Equation for a One-Dimensional Element

Next, to meet the equilibrium conditions, we derive the relationship between the nodal displacement and force for a one-dimensional element using the principle of virtual work given by Eq. (4.4).

We assume that no body forces act on element A. Then, the terms in Eq. (4.4) are given by

$$\begin{aligned} \int_{V^{(A)}} \{\sigma\}^T \delta\{\varepsilon\} dV &= \int_V (\delta\varepsilon_x^A)^T \sigma_x^A dV \\ &= \frac{S_A E_A}{l_A} \begin{Bmatrix} \delta u_1 \\ \delta u_2 \end{Bmatrix}^T \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} \end{aligned} \quad (4.27)$$

$$\int_{V^{(A)}} \{f\}^T \delta\{u\} dV = 0 \quad (4.28)$$

$$\int_{S^{(A)}} \{T\}^T \delta\{u\} dS = X_1^A \delta u_1 + X_2^A \delta u_2 = \begin{Bmatrix} \delta u_1 \\ \delta u_2 \end{Bmatrix}^T \begin{Bmatrix} X_1^A \\ X_2^A \end{Bmatrix} \quad (4.29)$$

Substituting Eqs. (4.27)–(4.29) into Eq. (4.4) and considering that Eq. (4.4) holds for δu_1 and δu_2 , we obtain

$$\frac{S_A E_A}{l_A} \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} \equiv [K_A] \begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} = \begin{Bmatrix} X_1^A \\ X_2^A \end{Bmatrix}, [K_A] = \frac{S_A E_A}{l_A} \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \quad (4.30)$$

Similarly, for element B, we obtain

$$\frac{S_B E_B}{l_B} \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \begin{Bmatrix} u_2 \\ u_3 \end{Bmatrix} \equiv [K_B] \begin{Bmatrix} u_2 \\ u_3 \end{Bmatrix} = \begin{Bmatrix} X_2^B \\ X_3^B \end{Bmatrix}, [K_B] = \frac{S_B E_B}{l_B} \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \quad (4.31)$$

$[K_A]$ and $[K_B]$ in Eqs. (4.30) and (4.31) are element stiffness matrices for elements A and B. The stiffness matrix is usually a symmetric matrix.

[Step 4] Obtain and Solve the Global System of Stiffness Equations

Noting that node 2 is shared by elements A and B, we assemble the stiffness matrices for the elements to obtain the global stiffness matrix.

First, we rewrite each of Eqs. (4.30) and (4.31) as

$$\begin{bmatrix} k_A & -k_A & 0 \\ -k_A & k_A & 0 \\ 0 & 0 & 0 \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \\ u_3 \end{Bmatrix} = \begin{Bmatrix} X_1^A \\ X_2^A \\ 0 \end{Bmatrix} \quad \text{where } k_A = \frac{S_A E_A}{l_A} \quad (4.32)$$

$$\begin{bmatrix} 0 & 0 & 0 \\ 0 & k_B & -k_B \\ 0 & -k_B & k_B \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \\ u_3 \end{Bmatrix} = \begin{Bmatrix} 0 \\ X_2^B \\ X_3^B \end{Bmatrix} \quad \text{where } k_B = \frac{S_B E_B}{l_B} \quad (4.33)$$

Adding the left-hand and the right-hand sides of Eqs. (4.32) and (4.33), we obtain the global stiffness equation as follows:

$$\begin{bmatrix} k_A & -k_A & 0 \\ -k_A & k_A + k_B & -k_B \\ 0 & -k_B & k_B \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \\ u_3 \end{Bmatrix} = \begin{Bmatrix} X_1^A \\ X_2^A + X_2^B \\ X_3^B \end{Bmatrix} \quad (4.34)$$

Elements *A* and *B* are connected at node 2 and the nodal forces must be balanced. Then, the following equation holds:

$$X_2^A + X_2^B = 0 \quad (4.35)$$

Here, we introduce the boundary conditions given to nodes 1 and 3. Node 3 is subjected to the external force X_3 , and $X_3^B = X_3$. Node 1 is fixed (boundary conditions), and therefore $u_1 = 0$. Then, Eq. (4.34) is rewritten as

$$\begin{bmatrix} k_A & -k_A & 0 \\ -k_A & k_A + k_B & -k_B \\ 0 & -k_B & k_B \end{bmatrix} \begin{Bmatrix} 0 \\ u_2 \\ u_3 \end{Bmatrix} = \begin{Bmatrix} X_1^A \\ 0 \\ \bar{X}_3 \end{Bmatrix} \quad (4.36)$$

Solving Eq. (4.36), we obtain the displacements u_1 , u_2 , and u_3 for all nodes.

Substituting them into Eqs. (4.25) and (4.26), we obtain the strain ε_x^A and the stress σ_x^A in element *A*. Also, substituting them into Eqs. (4.25') and (4.26'), we obtain the strain ε_x^B and the stress σ_x^B .

4.2 Methods for Structural Analysis

4.2.1 Modeling

It takes a significant amount of computation time to perform a fully three-dimensional analysis on a component. Therefore, simplified models are adopted when possible. For example, there are cases when a three-dimensional body can be treated as a two-dimensional one. Two-dimensional problems are indeed computed much more time-efficiently than three-dimensional problems (Fig. 4.4). If the boundary conditions are clearly defined, simplification methods can reduce the size of the region analyzed (Fig. 4.5), especially when a complex part of the shape is not affecting the overall equilibrium of forces [3]. With recent dimensional structure to be automatically meshed into small elements without developing any special model.

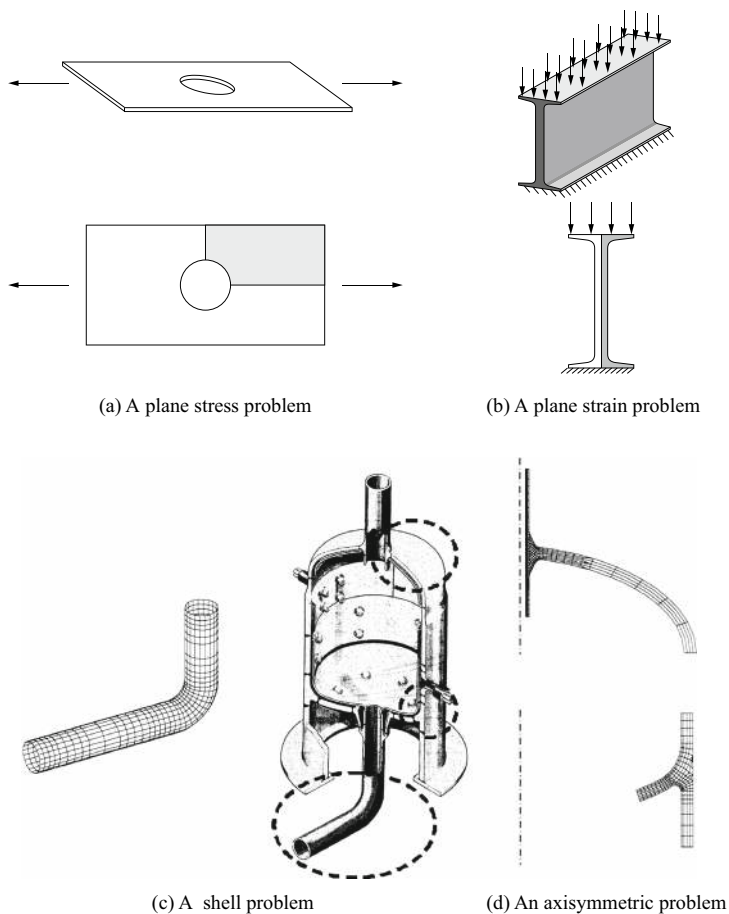


Fig. 4.4 Modeling by reducing the dimensions of the problem

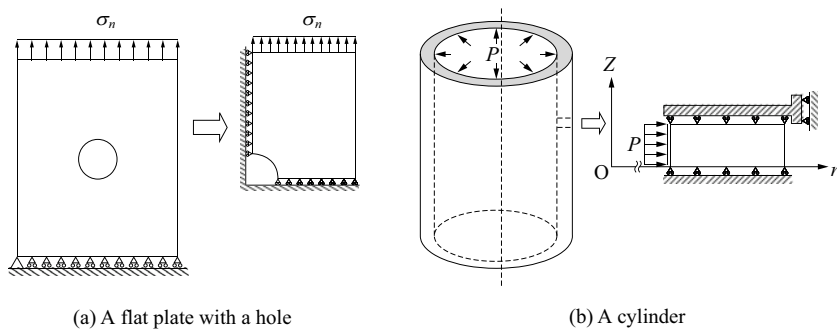


Fig. 4.5 Example of boundary conditions

4.2.2 Elements and Meshing

[1] Selection of elements

In the FEM, continuous structures are divided into one-dimensional bar elements, two-dimensional triangular or quadrilateral elements, or three-dimensional tetrahedral or hexahedral elements, and analyzed as assemblies of these elements. In general-purpose FEM codes, many types of elements with different characteristics can be used to provide flexibility in modeling structures. Therefore, the selection of an appropriate type of element is important in reducing the analysis cost and in obtaining results of required accuracy. Table 4.1 describes the classification of analysis problems and the types of elements used.

[2] Shape of the elements

It is desirable for higher accuracy that the shape of triangular elements resemble equilateral triangles and that quadrilateral elements are like squares. However, in

Table 4.1 Classification of analysis problems and elements used

Classification of problem		Type of element	Degrees of freedom of a node
Frame problem	Structure consisting of pin-connected members	Bar (truss) element	3 degrees of freedom of displacement
	Structure consisting of welded or riveted members	Beam (rigid frame) element	3 degrees of freedom of rotation in addition to 3 degrees of freedom of displacement
Plane problem	Thin plate (the stress in the thickness direction σ_z can be assumed to be zero)	Plane stress element	2 degrees of freedom of displacement
	Thick plate (the strain in the thickness direction ε_z can be assumed to be zero.)	Plane strain element	2 degrees of freedom of displacement
Axisymmetric problem	Axisymmetric solid body	Axisymmetric element	2 degrees of freedom of displacement
	Axisymmetric thin hollow body	Axisymmetric shell element	2 degrees of freedom of rotation in addition to 2 degrees of freedom of displacement
Three-dimensional problem	Three-dimensional solid body	Three-dimensional solid element	3 degrees of freedom of displacement
Plate/shell problem	Three-dimensional plate	Plate element	3 degrees of freedom of rotation in addition to 3 degrees of freedom of displacement
	Three-dimensional curved plate	Shell element	

practice, the entire analysis region cannot be divided into only equilateral triangles and squares. Division into shapes other than triangles and squares is allowed under certain restrictions. Figure 4.6 shows good and bad examples of use of triangular elements and four- or eight-node quadrilateral elements. In the auto-meshing of a three-dimensional structure, the element type used is in some cases limited to the tetrahedral element.

[3] Regularity of the mesh pattern

Meshing should be as regular as possible. However, in practice, due to the complexity of the shape to be analyzed or the need for changes in element size (depending on the location), meshing may have to be irregular. Figure 4.7 shows examples of allowable irregularities in mesh patterns.

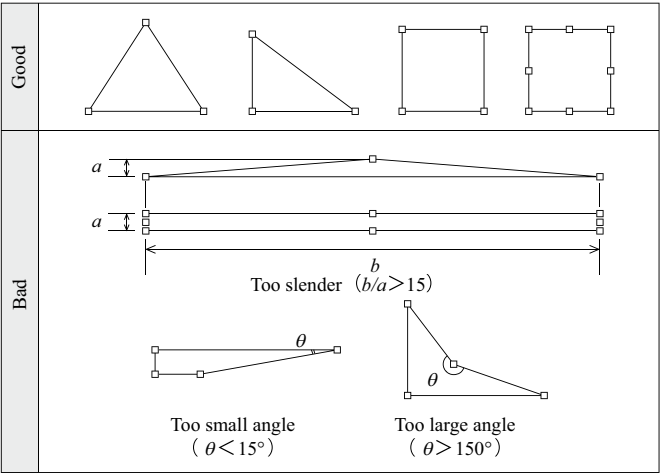


Fig. 4.6 Element shapes

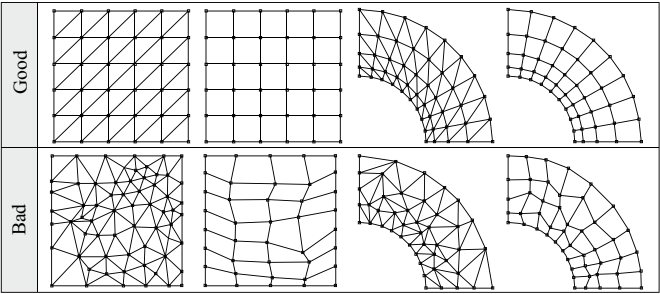


Fig. 4.7 Regularity of the mesh pattern

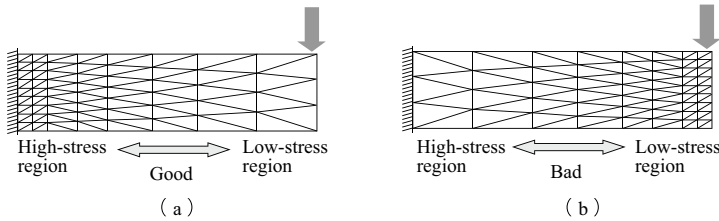


Fig. 4.8 Mesh size and anticipated stress

[4] Mesh size

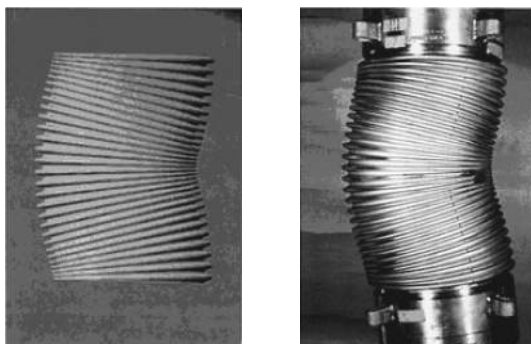
Excessively fine meshing is undesirable when aiming at reducing the calculation time and the amount of data generated. Generally, the required accuracy can be efficiently achieved by finely meshing regions with a steep stress gradient and coarsely meshing other regions, as illustrated in Fig. 4.8. The interface between fine and coarse elements is essential. If elements of extremely different sizes are directly connected, the overall accuracy of analysis decreases. Therefore, the mesh size should be gradually changed.

4.2.3 Verification of Analysis Results

FEM codes for elastic analysis are commercially available for use in calculation of designs. In the area of nonlinear analysis such as, elastoplasticity and creep, the FEM has been used as a complementary tool for research and development, as well as design. However, with rapid progress in computer performance and computational science, expectations are gradually increasing to employ nonlinear FEM as an alternative to costly experiment or as a direct tool to design structures. Commercial analysis codes, which source code is protected, are used recurrently. Analyses themselves have been increasingly outsourced to computer companies as well. However, in the case of reactor components, which require high reliability, the design engineers and researchers require a higher control on the numerical analyses and the results they generate must be thoroughly verified. Since the results of each analysis are affected by both analysis code and input data, a verification method depends on each specific problem.

A commonly recognized verification and validation approach (V&V) is described below. It includes quality assurance of the solution and has been certified by design engineers, and has been thoroughly discussed by academic societies and associations. The ASME Code in the US, which leads the development of quality assurance standards, proposes a V&V method aiming at reducing risk and achieving accountability, and is further developed in Ref. [4]. Finally, it is pointed out that the knowledge of strength of materials covered in previous chapters is essential to develop an appropriate model and judge the accuracy of the solution.

Fig. 4.9 An example of test-based verification (Analysis and test of internal-pressure buckling of a bellows performed by the Japan Atomic Energy Agency)



(a) Analysis

(b) Test

[1] Verification (checking the basic features)

First, the analysis formulas used in the code must be transcribed correctly in the code, and the overall code has to perform coherently, according to the formulas. For a simple problem, the analysis results should be compared with the theoretical solution (structural mechanics and strength of materials, etc.) Even for a complex problem, significant errors can be identified by this way. In addition, analysis of complex problems ought to be compared with results computed from a proven code. If such comparisons are not possible, then the analysis results have to be compared directly to experimental data. Figure 4.9 shows a comparison between analysis and test results on the buckling of a bellows obtained when no numerical analysis code was available for use as a reference tool. It is known that column-type buckling (Fig. 4.9b) occurs in a bellows with a large number of convolutions. This type of buckling happens due to the lateral deflection of the bellows being accelerated by the internal pressure, and it cannot be analyzed without considering the characteristics of the internal pressure following the deformation. Figure 4.9a shows a successful analysis performed by incorporating a load stiffness matrix to represent this effect.

[2] Validation (checking in an environment equivalent to the actual problem)

It should be checked whether reasonable results are obtained under a combination of environments close to the actual analysis environment and whether the meshing is appropriate. A trial-and-error approach, such as changing the model (such as the element mesh and type or the boundary conditions), is required to obtain a reasonable solution to a complex problem. Then, a comparison should be made with the results by other general-purpose FEM codes or by different analysis engineers. If there is no established method for solving the problem or the input data require validation, a comparison should be made with experimental results. However, usually, experiments are performed using small-scale models. Therefore, an overall judgment is often required to apply a scaling law to full-scale structures.

4.3 General-Purpose Structural Analysis Codes and Pre/Post-Processors

As shown in Table 4.2, many commercial general-purpose codes are available for structural analysis. These codes have the capability to perform thermal analysis, elastic analysis, elastic-plastic analysis, as well as vibration and impact analysis, which are all required for the design of reactor components. They also allow users to add their own evaluation methods and data using subroutine features.

Some codes have the capability to generate mesh and display analysis results. Dedicated pre/post-processors, such as FEMAP, PATRAN, or GiD, are commonly used to generate an analysis model and display the analysis results. These processors have a complete set of features, including capacity to read geometric data by CAD, automatic mesh generation, and numerical generation of graphic displays for results, at any cross section. In addition, they provide an interface with famous commercial codes shown in Table 4.2 and can therefore handle analysis data coming from different codes.

Figure 4.10 shows a typical flow of analysis using a general-purpose structural analysis code and pre/post-processors. When a dedicated pre/post-processor is used, an additional series of operations is performed by the processor software, which may significantly affect the total analysis time. Pre-processor data put into the analysis code are described below using simple models [5, 6] in Fig. 4.11 as examples. Table 4.3 shows an example of input data into FINAS. Other general-purpose codes have a similar configuration. If functions that are not supported by the pre-processor are used, the input data may be directly handled. Therefore, the format of the input data should be manually checked.

Table 4.2 The function of major general-purpose analysis codes

Analysis code		Function						
Name	Type	FEM-model generation	Temperature analysis	Structural analysis Elastic	Structural analysis Elastic-plastic	Structural analysis Vibration	Structural analysis Impact	Display of results
ABAQUS	Pre/analysis/post	○	○*2	○*2	○	○	○	○
ANSYS	Pre/analysis/post	○	○	○	○	○	×	○
FINAS	Analysis/post	△*1	○	○*4	○	○	×	○
I-DEAS	Pre/(analysis)/post	○	△*3	△*3	×	△	×	○
NASTRAN	Analysis	×	○	○	△	○	×	×
SAP	Analysis	×	○	○	×	○	×	×
LS-DYNA	Analysis/post	×	○	○	○	○	○	△

*1: Can model simple shapes.

*2: Temperature analysis can easily be linked to thermal stress analysis.

*3: Capabilities are not very high.

*4: Complete with elasto-plastic creep data on stainless steel

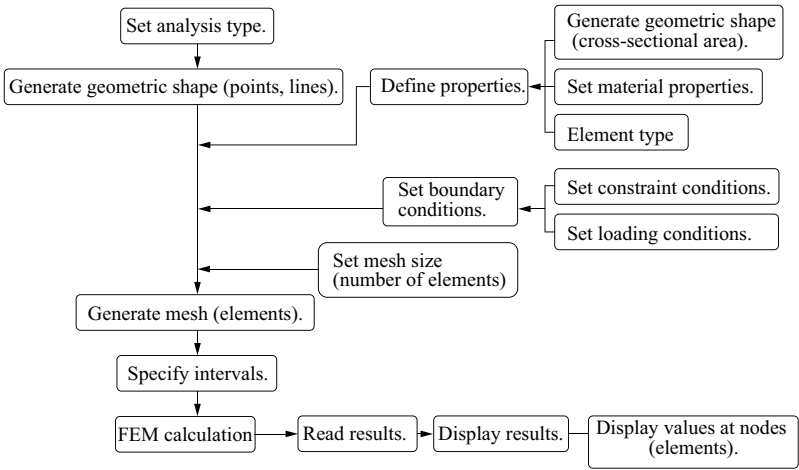


Fig. 4.10 A typical flow of analysis using a general-purpose structural analysis code and pre/post-processors

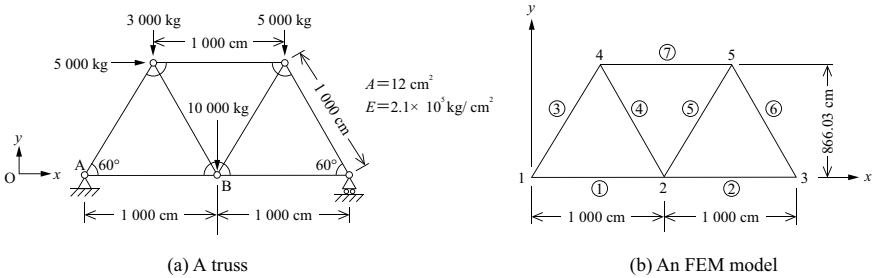


Fig. 4.11 Example of a simple FEM analysis

4.4 Example of Application to Practical Problems

This section describes an application to practical problems to show the analysis process and its important points. Let's consider the vessel in Fig. 4.12. Through the upper nozzle runs a flow of liquid sodium, alternatively hot and cold. The purpose of our analysis is to study the transient thermal stress created by the alternative flow. The FEM code used is FINAS [5] and the pre/post-processor used is FEMAP. The transient thermal stress analysis is performed in two steps as shown in Fig. 4.13. First, a non-steady state thermal analysis is performed to calculate the time history of the temperature distribution in the structure. Such analysis is achieved by inputting the varying fluid temperature obtained from thermal hydraulic analysis while considering the heat transfer coefficient as the thermal boundary conditions. Next, the temperature distribution in the structure at each time step is calculated from the thermal response

Table 4.3 Example of input data (FINAS)

FINAS									
TITLE		ELASTIC ANALYSIS OF A PIN JOINTED TRUSS							
STATIC		E							
MODEL									
NODE(Node information)									
1				0.0		0.0			
2				1000.0		0.0			
3				2000.0		0.0			
4				500.0		866.03			
5				1500.0		866.03			
ELEMENT TYPE(Element type)									
10	ROD2								
CONNECTION(Element information)									
1	10	20	30	1	2				
2	10	20	30	2	3				
3	10	20	30	1	4				
4	10	20	30	4	2				
5	10	20	30	2	5				
6	10	20	30	5	3				
7	10	20	30	4	5				
GEOMETRY(Geometric shape)									
20	1	12.0							
MATERIAL(Material properties)									
30									
	E			2.1E6					
BOUNDARY(Constraints)									
1									
		1			1	2			
		3			2				
		1	5		3				
FORCE(Loadng conditions)									
2									
		2					—10000.0		
		4				5000.0	—3000.0		
		5					—5000.0		
HISTORY(Loadng history)									
INTERVAL									
	BOUN	1							
	FORC	2							
END MODEL									
END FINAS									

analysis. This temperature distribution then becomes the input of the stress analysis model which calculates the thermal stress distribution and its time history. Thermal and thermoelastic analysis by the FEM is described in Chap. 6. Ref. [7] provides details of the analysis.

Considering that the top head and the nozzle are axisymmetric structures, and that the upper end portion of the vessels (top head) affects the stress in the nozzle, these structures can be modeled by axisymmetric elements as shown in Fig. 4.14. To allow the thermal transfer at the nodes, it is necessary to ensure consistency between the thermal and stress analysis models.

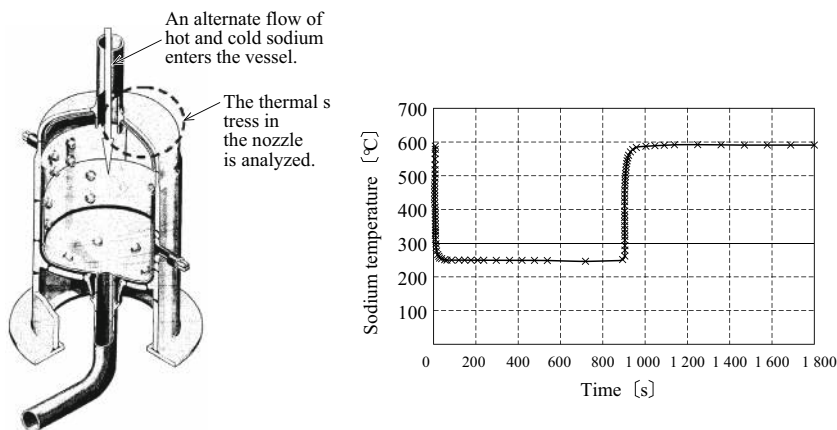


Fig. 4.12 Transient thermal stress analysis of a vessel subjected to an alternative flow of hot and cold sodium

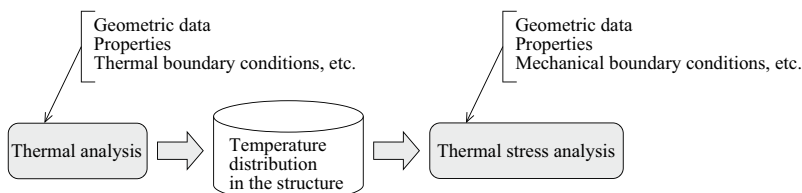


Fig. 4.13 Steps of the thermal and thermal stress analysis

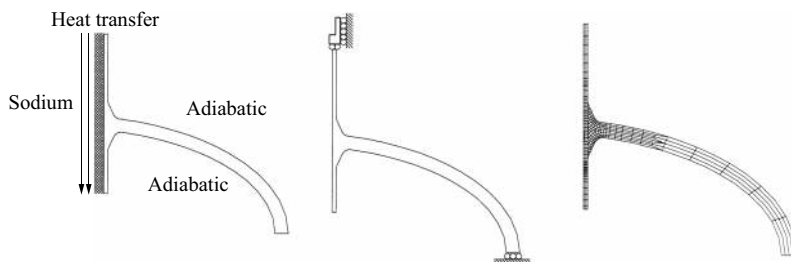


Fig. 4.14 The boundary conditions and the mesh model for the thermal and stress analyses

In thermal analysis, as the varying fluid temperature is determined with the heat transfer boundary conditions. The heat transfer coefficient at the boundary between the sodium and the structure and the change in sodium temperature must be provided as input data. They directly affect the results of analysis and therefore must be carefully specified. Also, it is necessary to input both thermal and mechanical properties as the material properties of the vessel. In the thermal analysis, constraints require

to be properly set, including the symmetry and the compatibility conditions. When generating the mesh model, the nozzle region of interest has to be finely meshed as shown in Fig. 4.14. The thermal and thermal stress analysis models using axisymmetric elements in Fig. 4.14 have 1626 nodes, 567 elements and 1626 degrees of freedom, and 1626 nodes, 493 elements and 3235 degrees of freedom, respectively. An analysis model is developed by viewing its computer graphic (CG) display repeatedly. The models in this example required a trial-and-error process of a few hours for completion. The computation time on a standard PC with 2.8 GHz single core CPU was 8.5 s for the transient thermal analysis in 123 steps, and 39.7 s for the thermoelastic analysis in the same number of steps. For a model like this, the computation time is much less than the time for generating input data.

The maximum stress and its location and time must be known to perform strength evaluation using calculation results. As shown in the output example in Fig. 4.15, these results are obtained by combining a CG display of temperature and stress distributions and a display of their time history. As described in Sect. 4.2.3, there are different ways to verify a solution. One simple way of checking is to approximate the values by substituting, for example, the temperature difference between the inner surface of the nozzle and the head plate into Eq. (6.8) in Chap. 6. Significant error can be avoided by doing so.

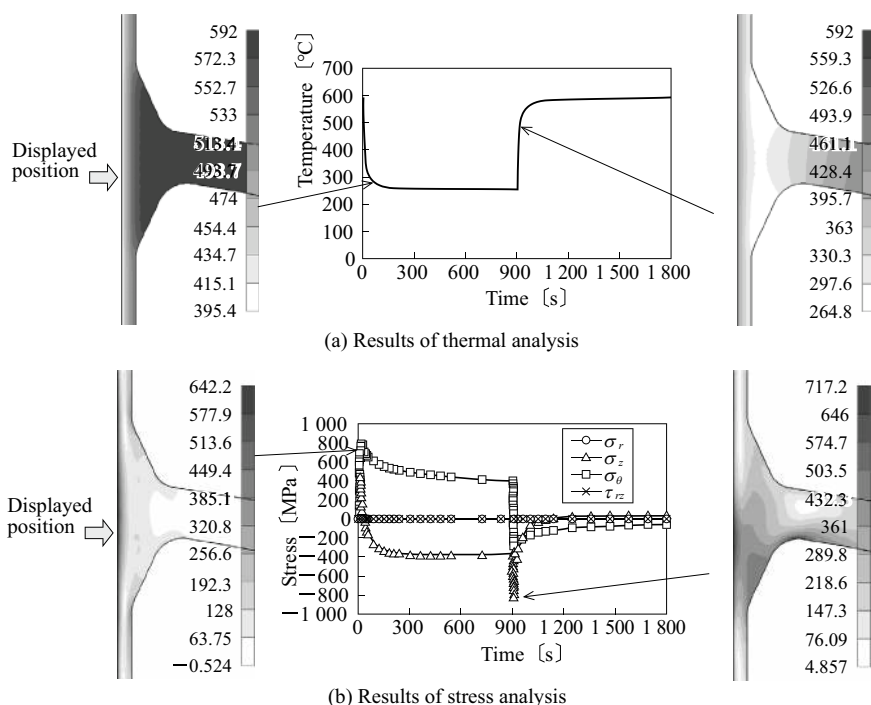


Fig. 4.15 An example of output of analysis results

4.5 Exercises for This Chapter

1. Name the three basic equations of structural mechanics. Which one of the three equations represents the principle of virtual work, which provides the basis for FEM structural analysis?
2. Describe the characteristics of a stress field in the following structures. Name the type of FEM element suitable for each of them.
 - (1) A thin plate subjected to an in-plane tensile load
 - (2) An H-steel bar subjected to a uniform axial load
 - (3) A thick cylinder subjected to an internal pressure
 - (4) A thin plate subjected to an out-of-plane bending load
3. Describe what you know about the verification methods for solutions of FEM analysis. Describe what you need to learn to acquire verification skills.

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Chapter 5

Structural Design of Light Water Reactors



Components of light water reactors, except for a small number of them, are used at relatively low temperatures and therefore do not undergo creep described in Chap. 3. This chapter describes primarily an essential design philosophy to assure the structural integrity of components used in the non-creep temperature range. The current design codes provide that the non-creep temperature range is below 375 °C for carbon steels and low-alloy steels and below 425 °C for austenitic stainless steels and nickel alloys.

The structural design of components involves several steps (processes). The process of thickness calculation called sizing and the process of demonstrating that the calculated thickness assures structural integrity are extensively described, with a particular focus on the design codes providing requirements and rules to be met to assure structural integrity. These design codes contain many concepts and terms probably unfamiliar to readers. These concepts are related to each other. In this chapter, the basic philosophy of the design codes, the concepts used, and detailed rules for applying the concepts to actual design projects are described in four steps, focusing on the understanding of the relationships among the concepts.

In the first step, Sect. 5.1 describes the basic philosophy of what design requirements must be met to assure structural integrity. In the second step, Sects. 5.2–5.5 describe the concepts in the design codes into which this philosophy is developed.

In the third step, considerations for transforming these concepts into stress evaluation in practice are described, and examples of such considerations are given. Section 5.6 describes a generally applicable, “design by analysis” approach, which is applied to pressure vessels. Piping systems are different from vessels in loading conditions and geometry (key loads and geometries of piping systems are limited, comparing to those of pressure vessels). These characteristics specific to piping systems are considered in the codes. Design codes for piping systems are discussed in Sect. 5.7. The approaches described up to Sect. 5.7 were established when strength of materials (or mechanics of materials) described in Chap. 2 was used for stress calculation before the common use of the finite element method (FEM) described in

Chap. 4. They are not necessarily based on the approach most suitable for FEM analysis. In recent years, with the recognition of the importance of an approach suitable for FEM analysis, such an approach has been developed and is incorporated into the design codes. The main concept of the approach is described in Sect. 5.8.

In the fourth step, Sect. 5.9 describes some important points of the practice of structural design, focusing on the rules of design codes into which the concepts are transformed. More specifically, the relationship with system design, specific rules of the design codes, and the design of welds requiring particular attention are described.

Concepts and considerations to be described up to Sect. 5.9 are so fundamental as to rely on, no matter what function or quality level is required to a component. For historical reasons, the content given in Sections up to 5.9 have been developed for application to components of “facilities for design basis events” that mean facilities to be designed to assure structural integrity for design basis events.” It may be noted here that such a design should be made according to the “worst case scenario” approach. As mentioned in detail in Sect. 1.5.2 [5] in Chap. 1, we have experienced three severe accidents so far. The latest is the Fukushima Daiichi nuclear accident in March 2011. Reflecting these experiences, measures against severe accidents or beyond design basis accidents have been legally required. In other words, it has been legally required to install facilities for coping with severe accidents. Therefore, in Sect. 5.10, design concepts for facilities for coping with severe accidents are explained, with referring to Japanese safety regulations.

It should be noted here that some of the content of this chapter is provided in Chaps. 1–3, but is described again and rearranged from the perspective of structural codes and for the convenience of the readers.

The above-mentioned introduction of design concepts for assuring structural integrity at the non-creep temperature regime is made, mainly referring to Refs. [1, 2].

5.1 The Basic Philosophy of the Current Design Codes

[1] The basic approach to assure structural integrity

Essentially, structures are fabricated using reliable technologies. Acceptance examination of materials and nondestructive examination of welds are strictly performed before use. Therefore, before use, they can be considered to have no harmful defects that can compromise their functions. Provided that the structure is free of defects, its structural integrity is assured by preventing potential failure modes with an adequate margin under the operating conditions assumed in design, including loading and environmental conditions. Since no harmful defect is assumed, this design concept is called “defect-free design” concept. Many boilers, pressure vessels, piping, and others that are fixed on the ground are said to be designed, fabricated, examined/inspected and so forth in accordance with this defect-free design concept.

As described in Sect. 1.3.2 in Chap. 1, the basic philosophy of failure prevention is to limit a load relative to the strength of a structure as shown by the following equation:

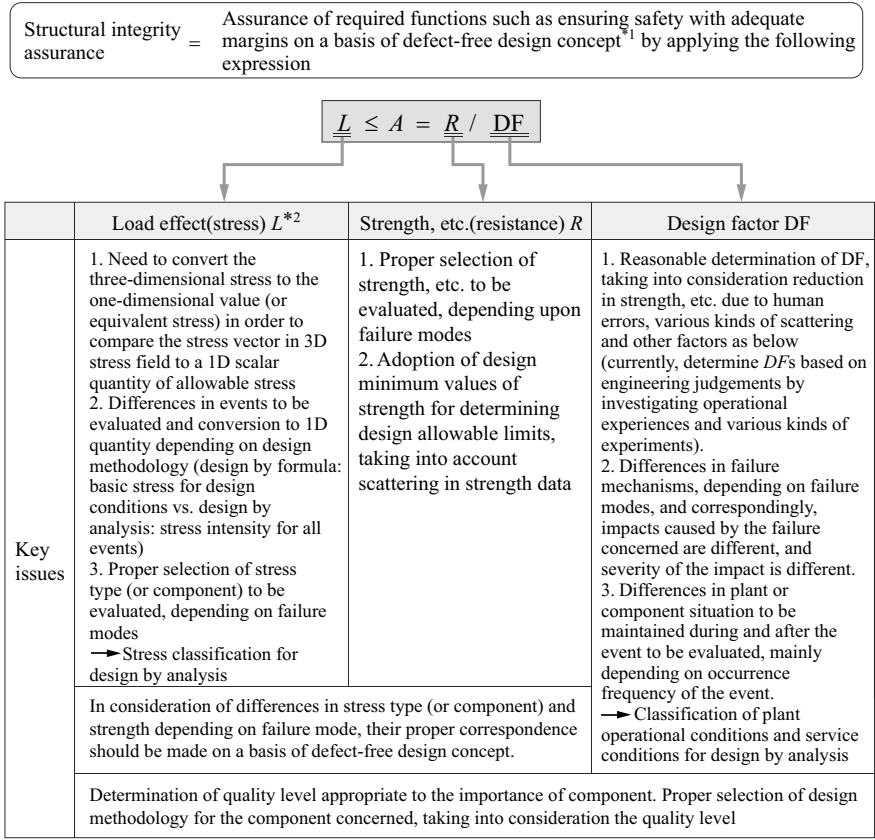
$$L \leq \frac{R}{DF} \quad (5.1)$$

L is the variable developed by the applied load and called the load effect. An example of L is stress. R represents strength or resistance to failure. A structure fails if loaded beyond the resistance. The equation means that when the stress reaches the resistance R , failure occurs and that the load effect is limited with a margin DF . The terms failure and fracture are differentiated. Fracture means that large cracks occur in a structure, literally leading to fracture. Failure means that the structure is excessively deformed, including fracture, and consequently cannot perform the required function.

Key issues to be discussed in developing Eq. (5.1) are summarized in Fig. 5.1. As mentioned at the beginning of this Chapter, Sections up to 5.5 provide details of these key issues and reasons why these should be discussed as the design concepts, and the following Sections give how these issues have been practically developed in design codes. The loading conditions of a structure are not as simple as can be described by a simple equation like Eq. (5.1). The structure can be subjected to three-dimensional loading or a combination of different types of loads that have different effects on failure. As a result of continued studies on how to apply the basic philosophy of Eq. (5.1) to various issues, the current design codes were established. The issues and their resolutions are described below, giving key considerations to the design factor DF . How each issue is addressed in the design codes is described in Sect. 5.2 and later.

Why in the first place do we need the margin DF ? Because it is not appropriate to allow the load effect as close as the strength of a material, considering that the strength varies with the fabricating conditions of the material, and structures may not be fabricated as designed. The material should be used with some margin for its strength. The margin DF called the design factor is introduced. The design factor used to be called the safety factor. It indicates how much the load should be reduced relative to the strength of the material to prevent failure, taking into account design errors and a possibility of lower value of material strength due to human errors, etc. than the design value, and has been empirically determined.

What you need to think is whether the value of DF should be the same for all cases. The answer is that the value is changed, taking into account various factors. The concept of DF is briefly described here, considering its importance. Details are described in Sect. 5.3. If the postulated failure mode causes a significant effect, the load effect is strictly limited relative to the strength of the structure due to the necessity of considering the possibility of such a failure. In other words, a large design factor is required. For example, if a catastrophic fracture called a “burst” occurs in which a crack is initiated in a vessel and the vessel is torn apart by fast growth of the crack, the pressurized fluid in the vessel bursts out, affecting a wide



*1) Historically, in several cases, flaws were developed due to a high cycle fatigue or stress corrosion cracking, etc. as a result of operation. Thus, during service, examinations or inspections have been conducted to detect such a flaw. If the detected flaw is judged to be harmful, repair or replacement may be done. Procedures for treating these flaws are provided in so-called maintenance codes. However, at the design and construction stage, the defect-free design concept may be applied.

*2) Stresses to be evaluated in design codes are calculated on the assumption of elastic material. This calculated stress is called elastically calculated stress, as mentioned several times in this chapter.

Fig. 5.1 Key issues in discussing structural integrity assurance by applying Eq. (5.1)

area. If there is the possibility of such a fracture, the possibility of L exceeding R must be reduced significantly. Normally in a fatigue failure, under cyclic loading, the crack gradually grows in the thickness direction and penetrates the wall of the vessel. If as the through crack further grows, certain conditions are met, the crack rapidly grows, causing a fracture of the vessel. Even in this case, at the time when the crack grows across the wall and an opening (or through-wall crack) is formed, the crack size may not reach the size of starting its fast or unstable growth, resulting in only a small leak of the fluid. In this case, time is available to detect the leak and take any action before a fracture or burst occurs. Such a situation is called Leak before Break

(LBB). It is not reasonable to provide the latter case with the same margin as for a burst fracture.

Another viewpoint is that in the case of a loading event that very rarely occurs, such as an accident, it is practical to allow to cause a plastic deformation to an extent that required functions are still kept so as to return to a safe plant state at the end of accident. For this case, an examination of whole areas may be required to check the reusability after the accident, and if needed, large-scale repairs will be required. On the other hand, for relatively likely events, such as loss of off-site power, it is not realistic to shut down the reactor for examination every time the event occurs, and it is desirable to restart the reactor operation without examination. For such events that the plant is restarted with no examination, it is necessary to strictly limit the allowable stress range within the stress range to cause no damage (practically, no plastic deformation). It is reasonable to limit the allowable range of load effect, according to the frequency of the event or the operating conditions of the plant.

It is ideal to reasonably determine the design factor, taking into account various factors, including the severity of the consequence of the failure, the occurrence frequency of the event, the condition to be maintained, and the allowable level of uncertainty. However, it involves difficult issues such as how to take into account human factors such as design errors and fabrication mistakes. In the current codes, the design factor has been deterministically determined based on experience and engineering judgment.

A wide spectrum of considerations must be taken into account in formulating Eq. (5.1). Basic considerations include:

- Understanding of types, causes (the driving force of failure, etc.), and mechanisms of failure: identification of the load effect (specifically, stress) and strength to cause each mode of failure, based on this understanding; and suitable classification of the load effect because the load effect to be evaluated depends on the failure mode under evaluation
- Determining a reasonable design factor: determining a design factor, taking into account the severity of the consequence of each failure mode; and reasonable relationships among the failure mode, the type of stress, and the design factor
- Classification of events (classification of operating conditions or service conditions), taking into account the occurrence frequency of the event and the state to be maintained after the event; and determining a design factor for each classified event
- Classification of quality levels, taking into account the difference in the importance of components in ensuring safety: correspondence between the classification of components based on the classification of importance or quality levels and the formulation of Eq. (5.1), in particular correlation with the design approach
- Formulation of Eq. (5.1), taking into account of the stress field applied: correspondence between the three-dimensional stress field applied to the structure and the allowable stress (a one-dimensional, scalar quantity) given by Eq. (5.1)

How these considerations are taken into account in the design codes and standards is described in Sect. 5.2. It is suggested to examine how what is described in Chaps. 2 and 3 is linked to them.

[2] Failure modes and their driving force (load or stress) (regarding L and R in Eq. (5.1))

In formulating Eq. (5.1), we must first investigate how a structure fractures or fails and what is the driving force of the fracture or failure. The basic concept of failure modes is described in Chap. 3. The failure modes are very important. Those assumed for the components of light water reactors in service at the non-creep temperatures are summarized below:

- (1) In a case of applying an external load such as a tensile mechanical load in a tensile test, the generated true stress increases as the load increases. Ductile fracture occurs when the true stress reaches the fracture strength (practically, the strain reaches the fracture strain), because such a kind of the generated stress increases in such a way as to meet the force equilibrium condition and further not to reduce its magnitude as a result of displacement. The fracture accompanied by a large amount of plastic deformation is termed the “ductile fracture.” Accordingly, a gross deformation precedes the ductile fracture. Under a loading condition of a bending moment combined with other external loads or under a pure bending moment loading, a “plastic collapse” will be caused as the failure mode, under which a gross distortion is developed in a major part of a structural element. Only one application of bending moment is also possible to cause the plastic collapse.
- (2) If cyclic loading is applied to the structure, loss of function due to progressive distortion or incremental collapse known as ratcheting should be considered. An excessive deformation may cause loss of a function (for example, heat removal of nuclear generated heat) required for equipment, even though the deformation does not reach the fracture deformation. The loss of required function other than that due to fracture is treated as a mode of failure. As is discussed later, a thermal stress does not cause the ductile fracture. However, a loading condition of a thermal stress combined with an internal pressure or a mechanical load may induce loss of a required function as a result of accumulating plastic deformations throughout a structure upon their cyclic application.
- (3) For cyclic loading, fatigue failure should also be considered. Most of the failures observed are of this type. Here, it should be mentioned that in this chapter, the “fatigue failure” is defined as an initiation of a fatigue crack, but not the fatigue fracture. At fatigue fracture, a load-carrying capability of a structural element is partly lost as a result of crack propagation. The above definition of fatigue failure is based on the defect-free design concept.
- (4) For compressive loading or external pressure loading, buckling should also be considered.
- (5) The fracture that is in contrast to ductile fracture is brittle fracture, which involves little plastic deformation. A typical example of brittle fracture is

a cleavage fracture, at which grains are separated along cleavage planes as observed in body-centered cubic (bcc) materials.

The above failure modes are expected to occur, common to equipment in service at a non-creep temperature regime, irrespective of operational and/or environmental conditions of industrial plants. On the other hand, failure modes expected to occur only in specific operational/environmental condition include stress corrosion cracking, environmental fatigue, hydrogen embrittlement, etc.

A structure is deformed when subjected to a mechanical load. Conversely, a deformation is capable to generate a load or stress. An example of the latter case is that a bar, which is deformed to some extent by heating (or thermally expanded), is restrained by a wall. The deformation, in this case the thermal expansion, may induce a compressive stress in this bar. Responses of the bar are illustrated in Fig. 5.2. As described in Sect. 3.3.1 in Chap. 3, there are various types of loads (for example, one is external loads such as internal pressure, and another is thermal loads). Different types of stresses occur in a structure depending on the type of applied load. It should be reminded that as described in Chap. 3, the fracture or failure mode depends on the type of stress. Plastic collapse and ductile fracture occur in such a case that the external load continues to increase and that correspondingly, the stress in the structure continues to increase to the fracture strength. Conversely, the load that causes plastic collapse or ductile fracture is an external load. The stress due to this load inherently increases with increasing external load. To be exact, the above stress is a type of stress that caused by a load whose magnitude is not reduced as a result of deformation. Internal pressure and gravity force are examples of such loads. This is termed the “load-controlled stress” (please refer to NB-3213.12 in Ref. [2]).

Next, investigate a type of stress that is caused by restraining a structural element by a wall, as shown in Fig. 5.2. Similar to Fig. 5.2, now, a compressive plastic deformation is assumed to be developed in a structural element. When removing a cause of the deformation (in the case of Fig. 5.2, return to the initial temperature), the element returns to its “initial” state. The “initial” state does not mean its initial shape, as clearly shown in Fig. 5.2, but means only the initial temperature state. At the “initial” state, the element is in compression due to the residual compressive plastic deformation. The size of the element is smaller than that of the initial shape. Again, impose the cause of the compressive plastic deformation. In the case of Fig. 5.2, it is the heating phase of the second cycle. In the last situation of imposing the cause (in Fig. 5.2, the point of maximum temperature), the element reaches the wall. This means that practically, no stress is induced at this situation. It is said relaxation of stress due to the residual plastic deformation. The magnitude of the load (in the case of Fig. 5.2, the push back load caused by the wall) is reduced for the second application.

More generally speaking, this type of stress is the stress caused in a structure by deformation-induced load generated, such as thermal load, at the constraint due to constrained deformation. In other words, if the deformation of the structure from any cause is constrained by the surrounding areas, the constrained portion of the structure is compressed (or stretched) and subjected to a load (caused because of

the deformation described above) from the surrounding areas corresponding to the amount of compression (or tension). As a result, stress occurs in the structure. This stress is not the stress equivalent to the deformation that would occur with constraint but is the stress equivalent to the amount of deformation that would occur with constraint minus the plastic strain after yielding. If the deformation that would occur with constraint is large, the structure yields, causing plastic strain. That is because

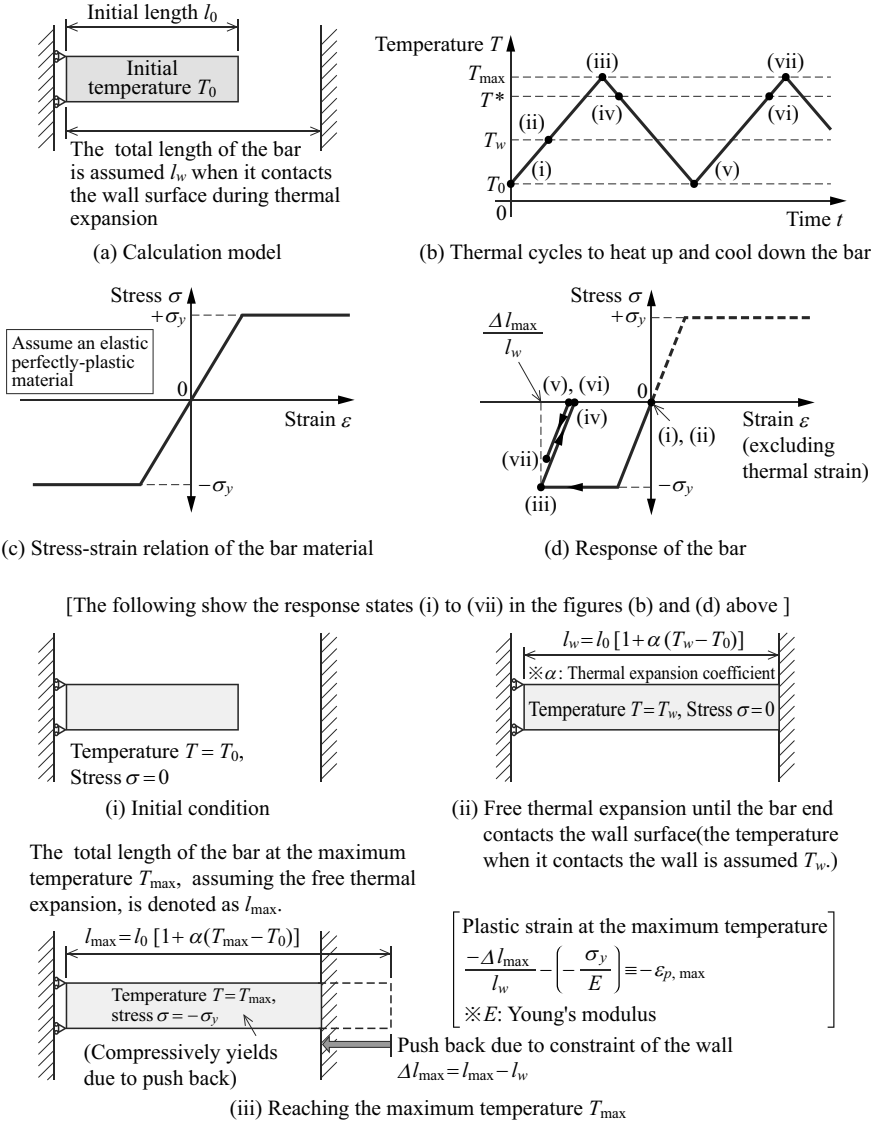


Fig. 5.2 Response of a bar subjected to heating and cooling (the bar at an end is not fixed)

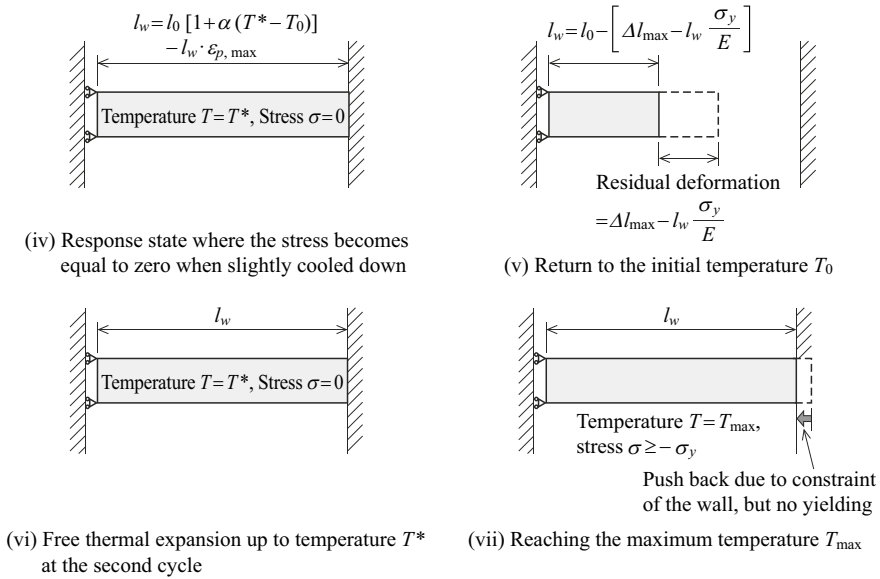


Fig. 5.2 (continued)

the sum of the strain (elastic strain) due to the stress and the plastic strain balances the constrained deformation. Therefore, with larger plastic strain, the stress is smaller for the same amount of the constrained deformation (for example, in a case of pre-existing plastic strain or residual plastic strain). In other words, the stress is relaxed due to plastic strain. As described in Sect. 3.3.1 in Chap. 3, this type of stress is termed “deformation (or displacement)-controlled stress.”

Load-controlled stress does not change unless the external load changes even if plastic strain occurs in the structure. Deformation-controlled stress changes with the plastic strain in the structure. Furthermore, as a result of plastic strain generated during loading in the first cycle, after the cause of deformation is removed, the plastic strain remains as residual strain until the next cycle of loading. When the same cause of deformation is applied, due to the residual strain, the constraint conditions are met under a stress smaller than in the first cycle. Deformation-controlled stress is relaxed when strain or deformation occurs that balances the constraint. A typical example of this is thermal stress. As suggested by this, deformation-controlled stress does not cause ductile fracture. A temperature increase of more than a few tens of thousands degrees is required to reach the strain to fracture (called ductility) in the first cycle, because fracture strains of many carbon steels exceed 50%, and those of austenitic stainless steels are larger than 100%. In reality, such a temperature increase never occurs. The fracture conditions cannot be met. However, ratcheting and fatigue failure in (2) and (3) can occur when cyclic loading is applied, and may induce an excessive deformation or develop crack initiation even though an accumulated strain

does not reach a fracture strain. In other words, those failures can occur due to deformation-controlled stress as well as load-controlled stress.

The important point mentioned in Chap. 3 is that the failure mode depends on the type of stress (or load). This necessitates classification of stress. A typical stress classification may be the load-controlled stress and deformation-controlled (or displacement-controlled) stress. However, it is not straightforward to correspond the failure modes to the above stress types. In order to make a clear-cut correspondence, it is better to adopt another stress classification, i.e., to classify the stress into primary, secondary or peak stress.

It should be noted here that fracture strains of brittle materials like ceramics are at most a few percent, so that there is likelihood to cause a fracture (or brittle fracture) even due to a thermal strain.

How the aforementioned issues are addressed in the design codes is described in Sect. 5.2.

[3] Types of stresses and the design factor

As described in [1] in Sect. 5.1, the severity of the failure modes described in [2] varies. The type of stress (load) that causes a failure mode varies with the mode. In Eq. (5.1), the load effect L varies with the failure mode. It is reasonable to determine the design factor DF , according to the severity of the consequence of the failure, for each failure mode and therefore for each load effect. The strength R also varies with the failure mode.

Section 5.2.1 in this chapter describes the classification of stress in relation to the failure mode and the strength involved.

[4] Operating conditions and service conditions

When the American Society of Mechanical Engineers (ASME) Boiler and Pressure Code (at that time, the Boiler Code) was established in the early twentieth century in the United States, the wall thickness of components has been calculated for a specified design pressure and temperature, using simple equations. This design approach is called “Design by Formula.” The pressure and temperature during an accident were not explicitly evaluated. From the 1950s to the 1960s, many nuclear power plants were built. In non-nuclear areas, the plant operating pressure was increased with the advent of super-critical pressure steam power plants. As a result, it became necessary to develop a design approach that is more reliable and allows a higher operating pressure (higher stress). Consequently, a “Design by Analysis” approach was introduced into the structural codes. This important design approach for enabling a higher reliability and higher allowable stress is explained below, comparing to the conventional “Design by Formula” approach.

In the “Design by Formula” approach, design conditions to specify a design pressure, design temperature and design mechanical load are defined so as to envelop the significant operating conditions. Then, for the specified design conditions, a wall thickness is calculated using a simple set of design formulae. Failures to be prevented were originally burst and ductile fracture, because this approach was proposed to

prevent explosions of boilers and other steam-powered components. Currently, failures by cyclic load applications (ratchetting and fatigue failure) are judged to be implicitly prevented by operational experiences.

The newly proposed “Design by Analysis” approach requires to prevent the potential failures by applying more strict methodologies, that is, requires to implement failure prevention assessments for all the events to be expected during a plant operation and further, explicitly for all modes of failure to be expected to occur during the operation. It should be noted here that regarding assessment for all the events, it is practical to classify all the events into several event groups and to select a representative event for each group, and then to conduct the assessment for the events selected. Further, it is reasonable that the allowable load range for likely events is strictly limited, while for very unlikely events, the load range in which the functions required to manage the event are properly achieved is reasonably determined, and is set as the allowable range. The event classification based on this approach was the classification of plant operating conditions because of focusing on plant-level functions. In the Japanese technical standards and codes, the plant operating conditions are classified into the Operating Conditions I to IV and the Testing Condition (or Test Condition) for facilities for design basis events. The terms of this classification are different from those in ASME Boiler and Pressure Vessel Code, although the content for terms of both codes is identical. However, as described later, in some cases, the functions required for each operating condition were different from those required for components, and confusion could be created. Event classification as the classification of service conditions was developed from the standpoint of component design and has been used to date. The service condition classification is as follows: the Service Conditions A to D and the Testing Condition (or Test Condition). The detailed definition of the classification of operating conditions and classification of service conditions is described in Sect. 5.4.2 for facilities for design basis events.

Thus, the structural response to be maintained by the components depends on the operating condition or service condition. In structural design, the design factor in Eq. (5.1) varies according to the operating condition or service condition.

[5] Classification of components by importance

The classification (or type) of components is another important consideration in formulating Eq. (5.1). The importance of the consequence of the failure of the postulated mode depends on the type of stress as discussed previously. Furthermore, it strongly depends on the importance of the component. A comparison between a component containing no radioactive materials and the reactor pressure vessel will help understand this. The importance of a component can be described by the level of quality to be achieved. The required level of quality of each component is determined in a comprehensive manner based on the level of quality required to ensure safety and other functions. Components, therefore, can be divided into several groups according to the required level of quality, that is the order of importance. For components of extremely high importance or those requiring an extremely high level of quality, such as a reactor pressure vessel, the design by analysis approach is applied, and structural integrity for all the potential failure modes is evaluated in the design stage.

In the fabricating stage, for example, only highly reliable weld geometries such as a full penetration geometry of welded joint are used and strict weld examinations such as a combined application of volumetric examination (e.g., radiographic examination) and surface examination (e.g., liquid penetration examination) are performed. Thereby, the extremely high level of quality required is achieved.

As a conclusion, selection of design approach, welding method, nondestructive examination and so on should be appropriately made by considering the importance of component. This means that component classification according to its importance is of great importance as a structural design. Section 5.5 describes the principle of the classification of components of facilities for design basis events, while that of facilities for coping with severe accidents, etc. is given in Sect. 5.10.

[6] Design approach

The factors that change the value of DF in formulating Eq. (5.1) include the design approach, as well as those mentioned above. The following three approaches are available to design the configuration and dimensions of a structure to assure its structural integrity under the applied load:

- (a) Design by Formula or Design by Rule
- (b) Design by Analysis
- (c) Design by Experiment.

In design approaches (a) and (b), stress is calculated and design is carried out to meet Eq. (5.1). The design factor DF depends on the design approach selected, roughly speaking, the level of detail of the evaluation of stress due to the applied load. In design approach (c), a test to fracture or to a loss of required function is performed using a full-scale model to determine the allowable operating conditions (for example, the maximum operating pressure).

Figure 5.3 shows how different “Design by Formula” and “Design by Analysis” are in design flow, required information, and wall thickness calculation and stress evaluation. This figure illustrates design flows for components of facilities for design basis events. Of course, the philosophy underlying this figure is applicable to those for facilities for coping with severe accidents, etc. In “Design by Analysis,” it is required to assess the structural integrity for all potential failure modes and for all design basis events (actually, as mentioned above, for the selected representative events). Hence, events shall be classified into service conditions and operating conditions; information is required on the stresses due to each event; each stress is classified (or divided into stress components) taking into account its relationship with each failure mode; and based on these information, sizing (or determination of thickness) and other integrity evaluations are implemented so as to satisfy allowable limits. Then, proceed to the detailed design for manufacturing.

It should be noted that Fig. 5.3 does not necessarily show the whole picture of the design process. It is suggested to compare Figs. 5.3, 5.4, 5.5, 5.6, 5.7, 5.8, 5.9, 5.10, 5.11, 5.12, 5.13, 5.14, 5.15, 5.16, 5.17, 5.18, 5.19, 5.20, 5.21, 5.22, 5.23, 5.24, 5.25, 5.26 and 5.27 to understand the whole picture of the design process and the purpose and role of each step shown in the whole picture.

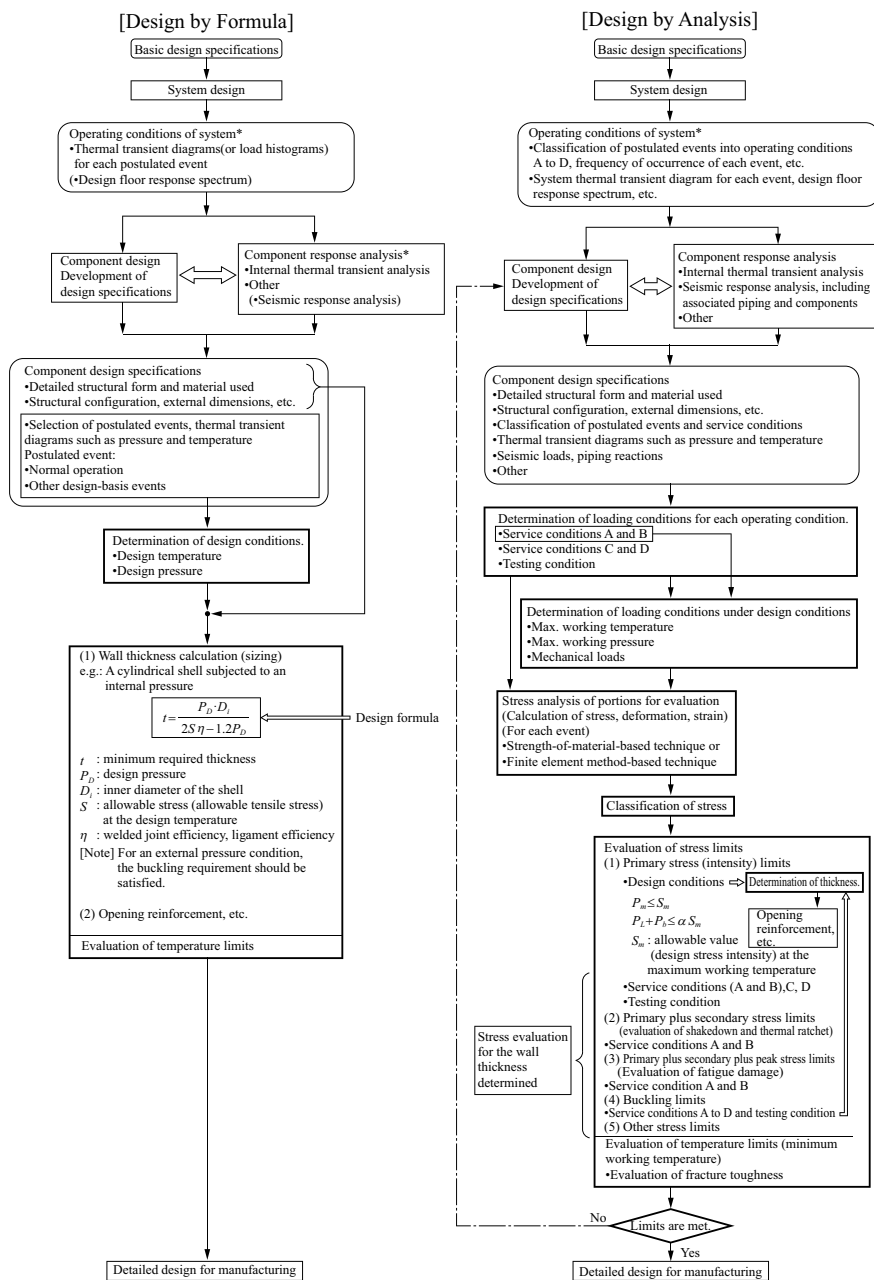


Fig. 5.3 A comparison of the design system between “Design by Formula” and “Design by Analysis”

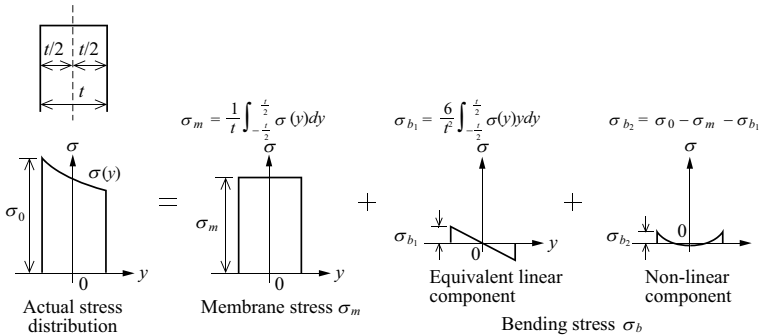


Fig. 5.4 Primary membrane and bending stresses (in the general region)

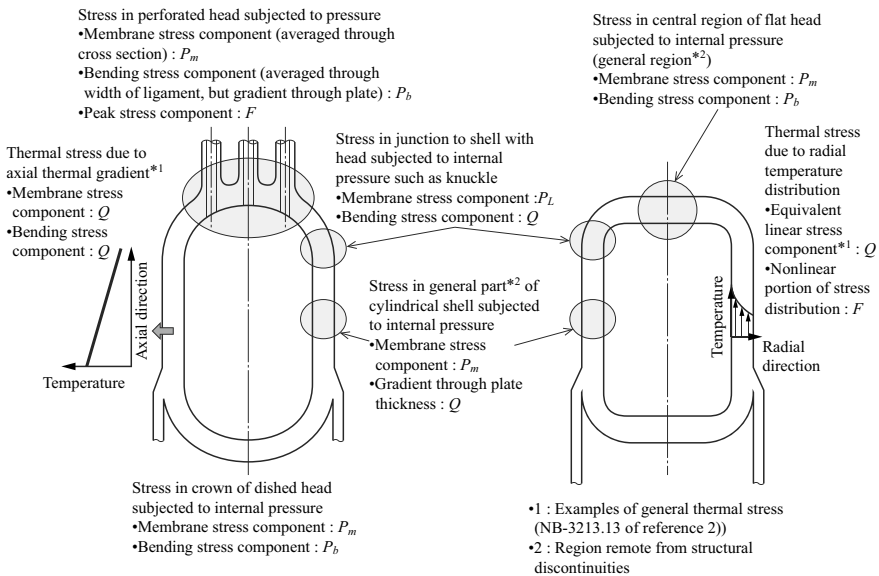


Fig. 5.5 Examples of stress classification in vessels

“Design by Formula” (hereinafter, call the Design by Formula approach. Similarly, “Design by Analysis” to the Design by Analysis approach) shown on the left side of Fig. 5.3 is used in many codes and standards including the ASME Boiler and Pressure Vessel Code Section I and Section VIII Division 1 (Pressure Vessel Code). In this design approach, design conditions (design temperature and pressure, etc.) are determined to include significant service conditions. In Japanese standards, terms relating to the design condition are different from the ASME Pressure Vessel Code: the maximum allowable working temperature and pressure. The wall thickness of a structure is determined to keep the basic stress (normally, circumferential stress)

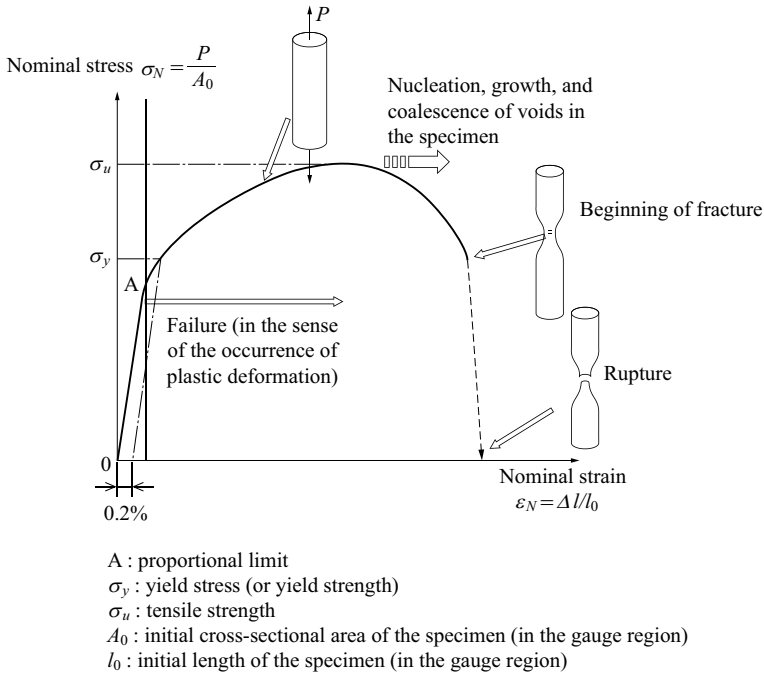


Fig. 5.6 Failure, fracture, and rupture of tensile test specimen

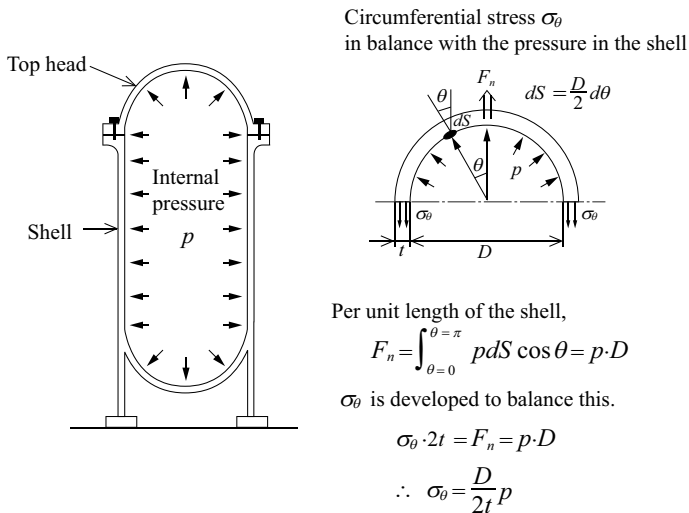


Fig. 5.7 Stress in the shell of a cylindrical vessel subjected to an internal pressure

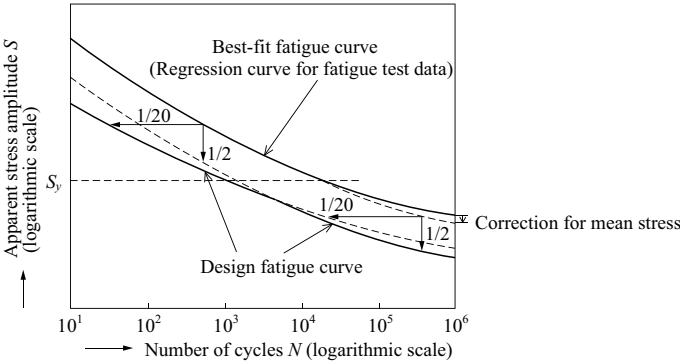


Fig. 5.8 Design factor in a design fatigue curve (*S-N* curve)

Plant operational state (defined for safety design)	Plant operating conditions (defined for structural design)	Classification of facility and equipment									
●Normal operation ●Anticipated operational occurrence ●Design basis accident	●Operating condition I ●Operating condition II ●Operating condition III ●Operating condition IV ●Testing condition	Facilities for design basis events Reactor pressure vessel Reactor containment vessel Facilities for coping with severe accidents, etc.	[Classification of facility and equipment for facilities for design basis events] (The following are defined in the regulation*) <table><tr><th>Facility classification</th><th>Equipment classification</th></tr><tr><td> ●Safety facilities - Facilities important to safety - Engineered safety features ●Seismically important facilities </td><td> ●Equipment for coping with design basis accidents </td></tr></table> [Classification of facility and equipment for facilities for coping with severe accidents, etc.] (The following are defined in the regulation*.) (The classification is made on the basis of two different criteria: (accident prevention/accident mitigation, and permanent type/ portable type)) <table><tr><th>Facility classification</th><th>Equipment classification</th></tr><tr><td> ●Specialized safety facilities </td><td> ●Equipment for coping with severe accidents, etc. - Equipment for preventing severe accidents ●Seismically important and permanent equipment for preventing severe accidents - Equipment for mitigating severe accidents - Permanent equipment for coping with severe accidents, etc. - Portable equipment for coping with severe accidents, etc. </td></tr></table>	Facility classification	Equipment classification	●Safety facilities - Facilities important to safety - Engineered safety features ●Seismically important facilities	●Equipment for coping with design basis accidents	Facility classification	Equipment classification	●Specialized safety facilities	●Equipment for coping with severe accidents, etc. - Equipment for preventing severe accidents ●Seismically important and permanent equipment for preventing severe accidents - Equipment for mitigating severe accidents - Permanent equipment for coping with severe accidents, etc. - Portable equipment for coping with severe accidents, etc.
Facility classification	Equipment classification										
●Safety facilities - Facilities important to safety - Engineered safety features ●Seismically important facilities	●Equipment for coping with design basis accidents										
Facility classification	Equipment classification										
●Specialized safety facilities	●Equipment for coping with severe accidents, etc. - Equipment for preventing severe accidents ●Seismically important and permanent equipment for preventing severe accidents - Equipment for mitigating severe accidents - Permanent equipment for coping with severe accidents, etc. - Portable equipment for coping with severe accidents, etc.										
●Severe accident, etc. ●Beyond design basis accident with the possibility of escalating to severe accident ●Severe accident											

*The “Ordinance on Standards for Location, Structures and Equipment of Commercial Nuclear Power Reactors and their Auxiliary Facilities”

Fig. 5.9 Classification of facilities for design basis events and facilities for coping with severe accidents, etc.

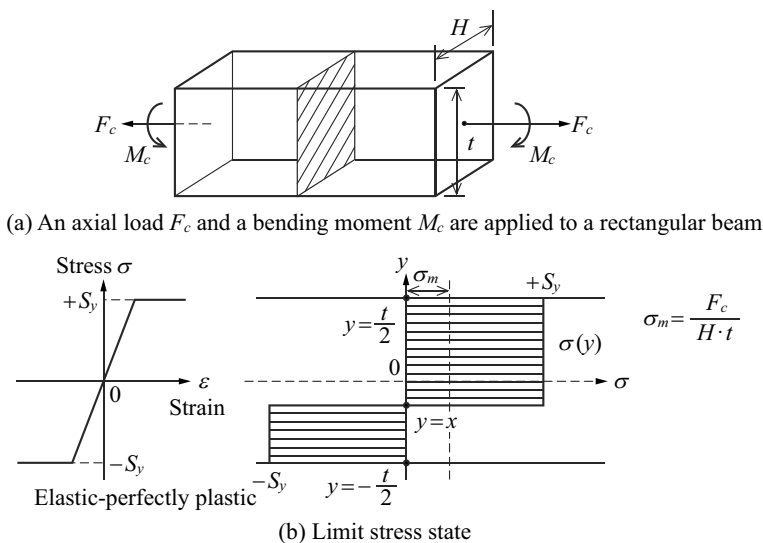
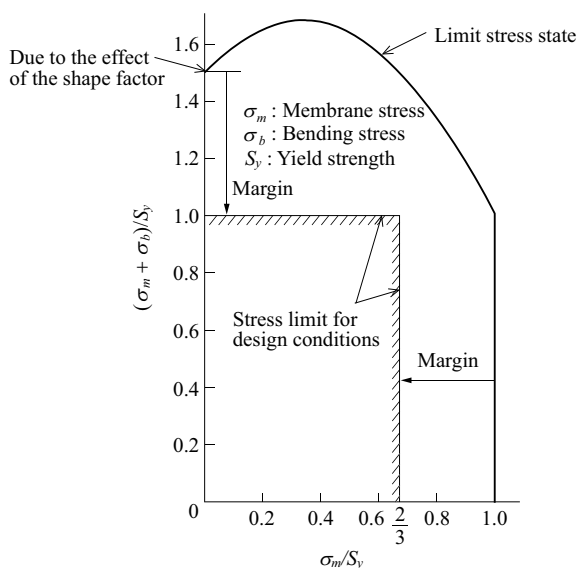


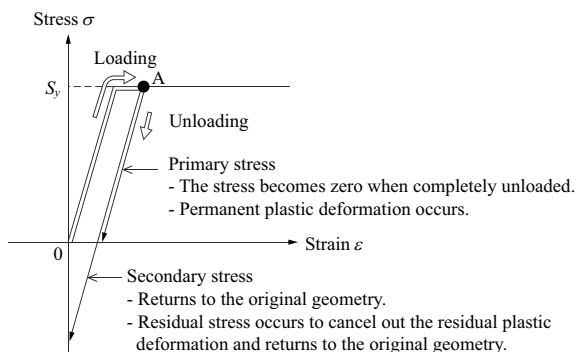
Fig. 5.10 Limit stress under membrane and bending stresses

Fig. 5.11 General primary membrane plus bending stress limit for a rectangular beam (The model and the limit stress state are shown in Fig. 5.10)



due to the design pressure below the allowable stress (R/DF on the right-hand side of Eq. (5.1)). It is determined that structural integrity is assured with this thickness. Evaluation of thermal stress, stress concentration at structural discontinuities, and other details is normally not required if a well-established structural form or configuration is used. This is because experience suggests that structural integrity is assured

Fig. 5.12 Difference in primary and secondary stresses: residual stress and permanent plastic deformation during unloading in the material subjected to loading beyond the yield stress



without explicitly evaluating thermal stress and stress at structural discontinuities if the value of the design factor DF used to determine the allowable stress in the Design by Formula approach is used. In fact, based on failure reports, this value of DF has been used as the value that does not cause failure under operating thermal conditions or at structural discontinuities. You may think that the wall thickness is calculated only for the design pressure and therefore failure modes such as fatigue and thermal ratchet are ignored, but it is not correct. The Design by Formula approach is based on experience. If an established structural form and configuration and operating conditions are used, structural integrity is assured. Conversely, the Design by Formula approach should not be used if the structural form and configuration and the operating conditions are outside the applicability of the approach.

However, plant components requiring an extremely high reliability and subjected to severe thermal stress, such as reactor pressure vessels of nuclear power plants have emerged. In order to assure the structural integrity for such situations, it was required to adequately assess the integrity for failures to be caused by cyclic applications of thermal stress under combined conditions with mechanical load stress, the integrity at structural discontinuities and so forth. Further, plants in service at high pressures such as super-critical pressure have emerged. The higher the operating pressure, the thicker the wall thickness. Hence, fabrication technologies applied were approaching to their limits. Thus, it was desired to reasonably reduce design factors with keeping the same safety level as that required for the Design by Formula approach. Summarizing the above, it was required to adequately assess such modes of failure as a low-cycle fatigue, for which the Design by Formula approach did not explicitly assess. Further, new types of plant components have emerged, whose operating conditions are beyond the applicability of the design by formula approach, and their structural configurations are outside the range of operating experiences. From such backgrounds, an alternative design approach was proposed that can handle severe operating conditions and allows the use of a higher allowable stress (reduced DF) while achieving the same level of safety as is achieved by the Design by Formula approach. In the latter of 1950s to 1960s, it was standardized as the Design by Analysis approach.

The Design by Analysis approach was established as a general design approach in Division 1, Subsection NB applicable to reactor coolant boundary components,

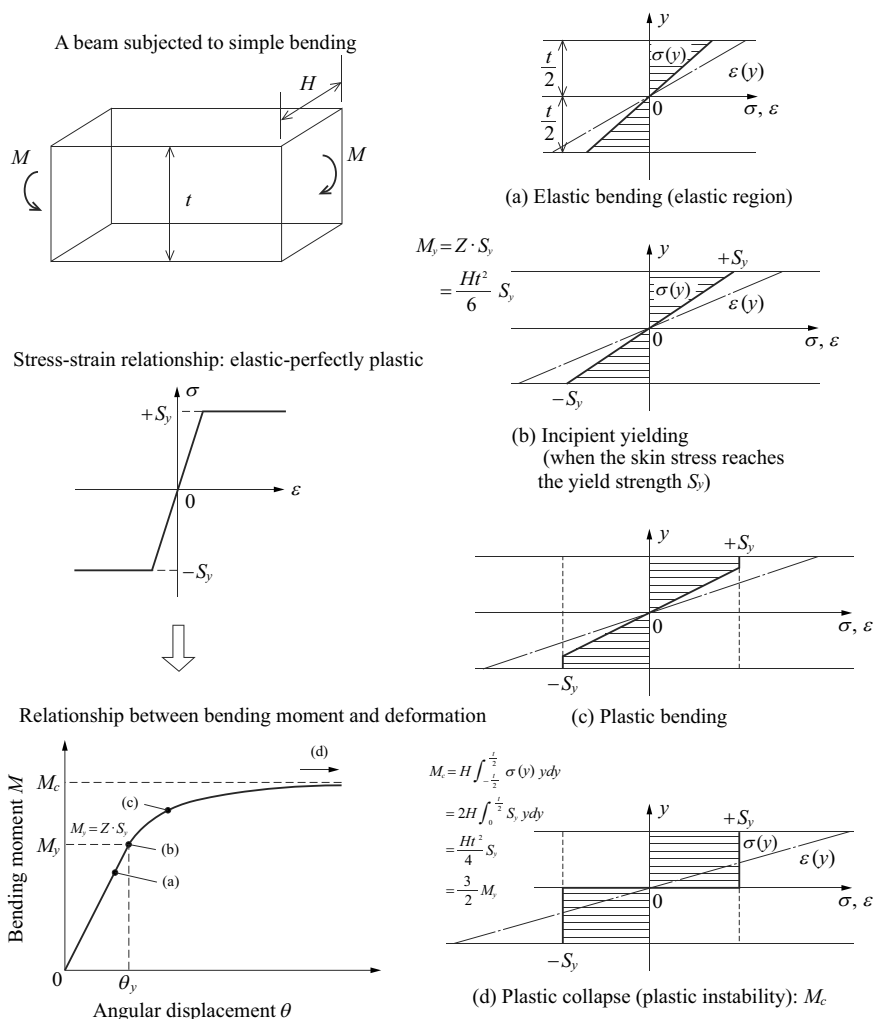
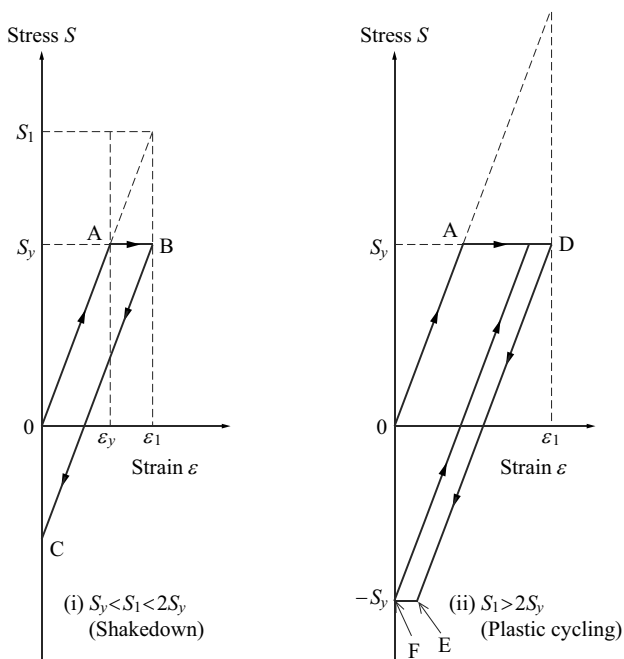
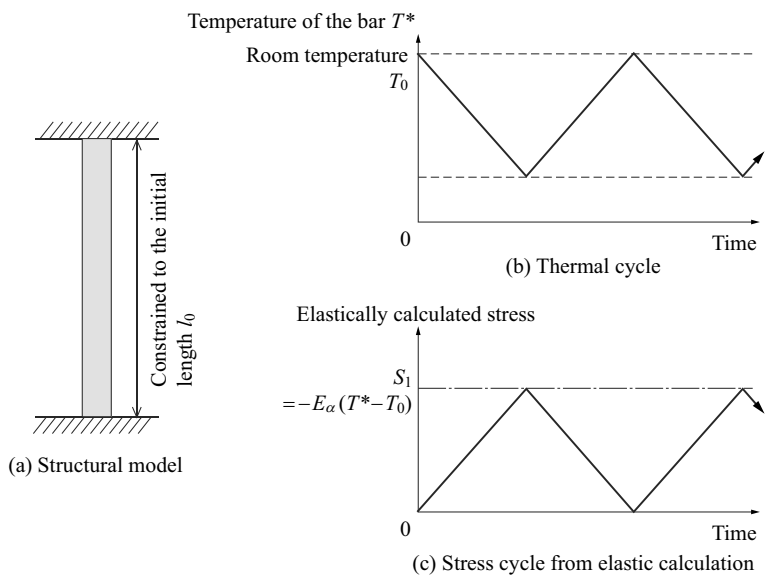


Fig. 5.13 Plastic collapse of a beam subjected to a bending moment

particularly vessels (Class 1 vessels) in the ASME Boiler and Pressure Vessel Code Section III. In non-nuclear areas, the approach is established as the ASME Boiler and Pressure Vessel Code Division 2 as an alternative to Section VIII Division 1 described above. Details are described in Sect. 5.2 and later. As shown on the right side of Fig. 5.3, stress limit evaluation for all of the potential failure modes are explicitly performed. A detailed stress analysis is performed for loading conditions under various events (categorized as service conditions A to D and testing condition in Fig. 5.3), and the components of generated stresses are classified (classification of



(d) Response of the bar of elastic-perfectly plastic material

Fig. 5.14 Stress-strain behavior when yielding occurs

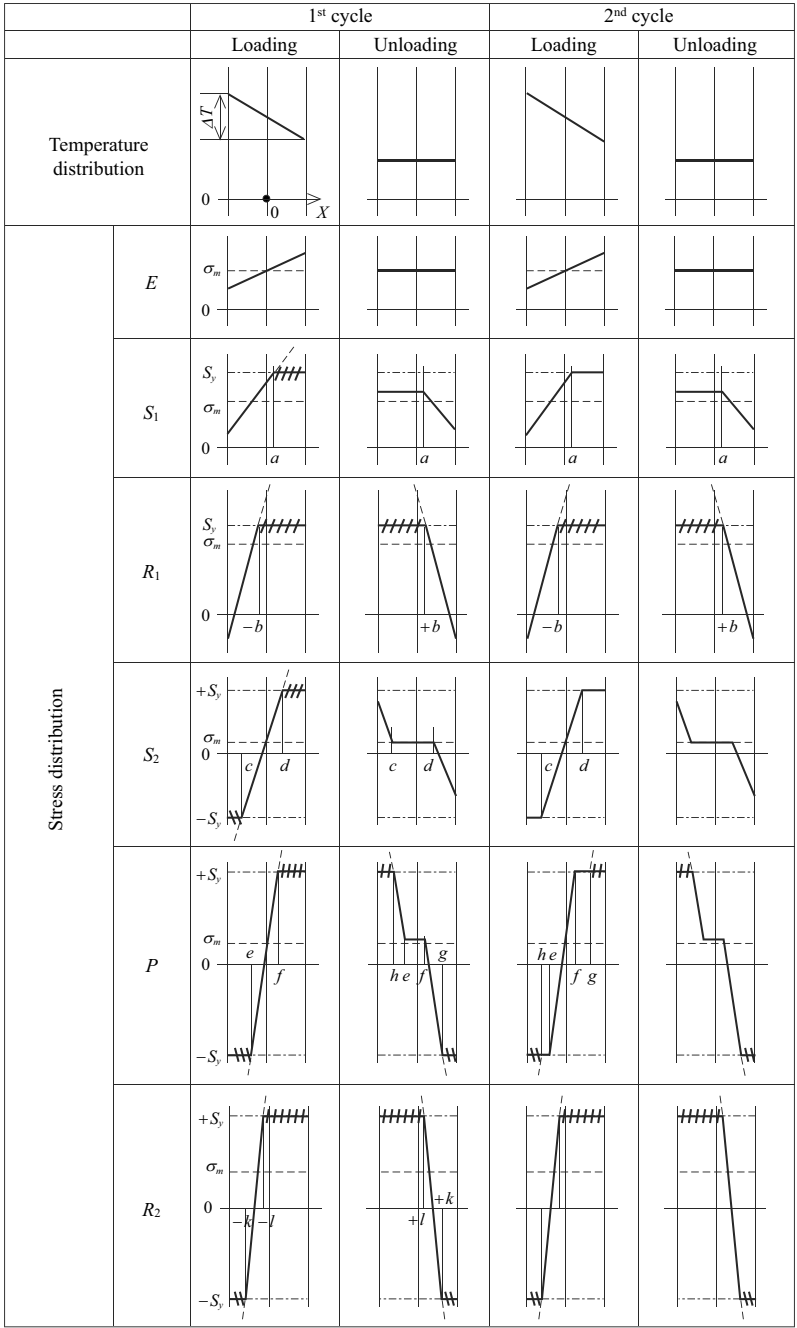


Fig. 5.16 Classification of stress distributions in the wall of cylinder subjected to sustained primary stress σ_m and cyclic thermal stress σ_t

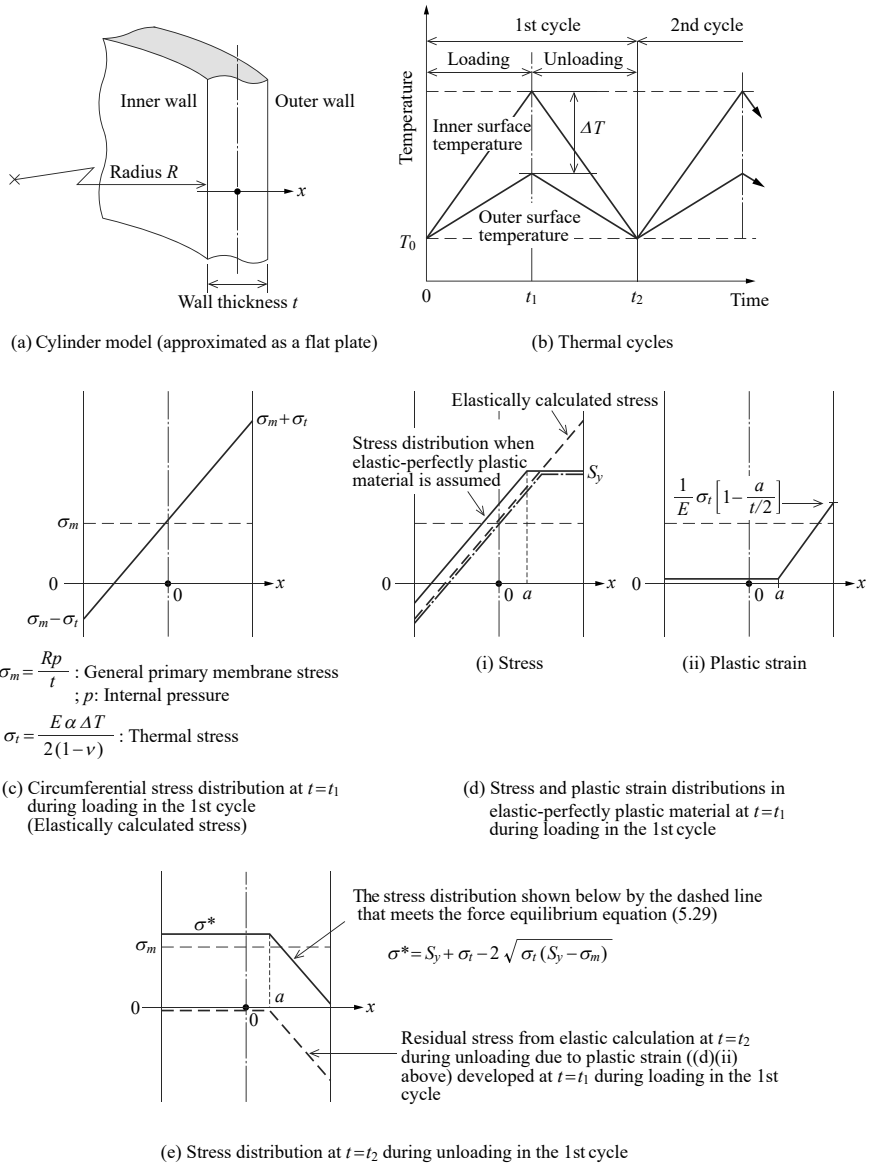


Fig. 5.17 Detailed stress and plastic strain distributions in shakedown behavior S_1

helping conceptually understand a practice of wall thickness calculation. From this calculation, readers can understand thinner wall thickness by applying the Design by Analysis approach than that by the Design by Formula approach, because of lowering the design factor from 3.5 for the Design by Formula approach to 3.0 for the Design by Analysis approach, by which S_u is divided to derive the allowable stress.

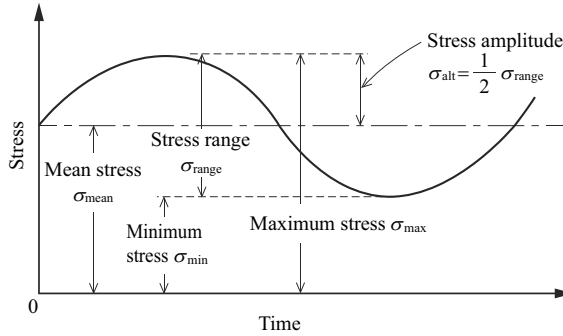


Fig. 5.18 An example of a stress cycle in elastic analysis

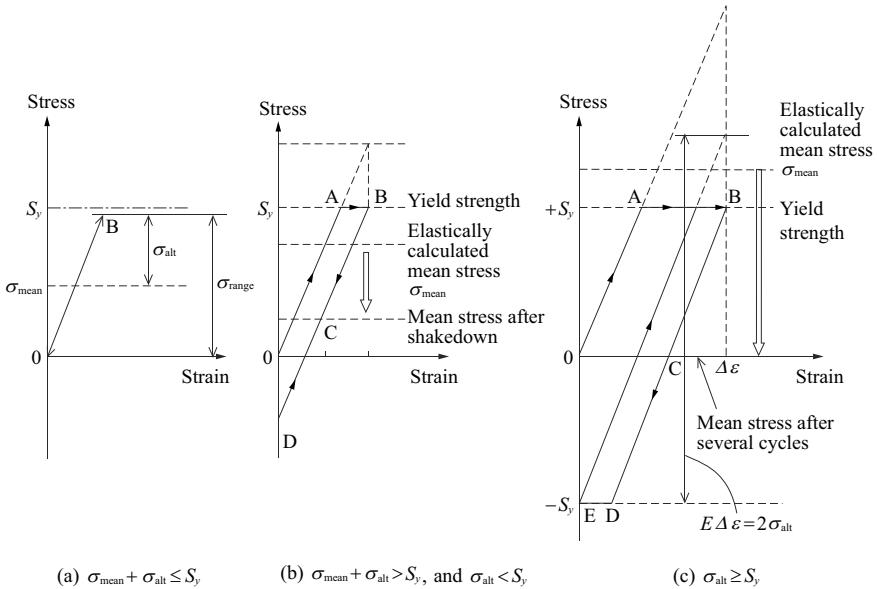


Fig. 5.19 Shift of mean stress due to yielding

However, reduced DF and the increased allowable stress result in an increased probability of structural fracture of a cylindrical shell, with a smooth surface, subjected to the rated pressure. Therefore, as described in detail in [1] in Sect. 5.3.2, if the Design by Analysis approach is used to apply a design factor smaller than that obtained by the Design by Formula approach, fatigue analysis and notch toughness testing, etc. are performed. In addition, material acceptance examination, weld design, and nondestructive examination, etc. are strictly performed. These strict requirements (the details are given later) are imposed in order to reduce a probability of component fracture by lowering a probability of defect.

Fig. 5.20 Shift of mean stress component due to yielding

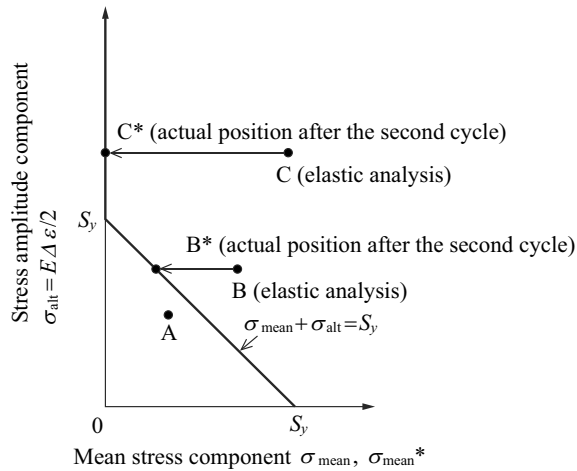
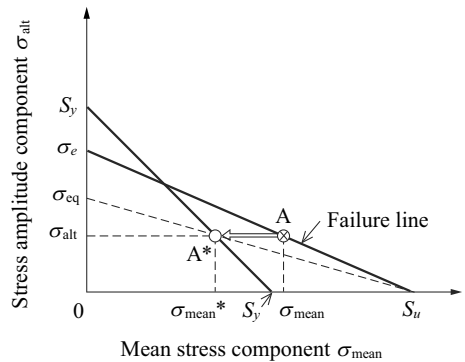


Fig. 5.21 Calculation of equivalent stress amplitude σ_{eq} (as zero mean stress) due to mean stress



There is a large difference between the Design by Formula approach and the Design by Analysis approach. Sufficient attention should be paid to which design approach is used in the design codes and standards to be used. In the current design codes for nuclear facilities, either the Design by Analysis approach or the Design by Formula approach is used depending on the importance of the component and further on the type of component (or, even in the same class of component, the design approach changes depending on the type(vessel, piping, pump, valve, support)). Table 5.2 shows which design approach is used for which nuclear power plant components and supporting structures. The table shows the case of the rules on design and construction [1] issued by the Japan Society of Mechanical Engineers (JSME). The Design by Analysis approach is applied to class 1 vessels such as a reactor pressure vessel, class 1 piping, a part of class 1 valves (especially, the valve body), and reactor core supporting structures (or, simply, reactor core supports). The Design by Formula approach is applied to the other components and supports. As special cases, class 1 pumps, reactor containment vessels (class MC), class 1 supports, class 2 vessels

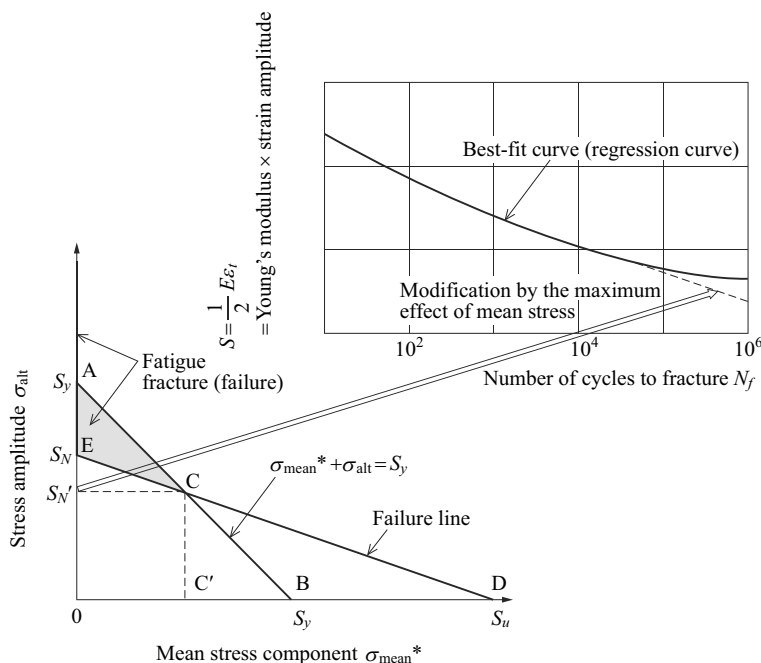
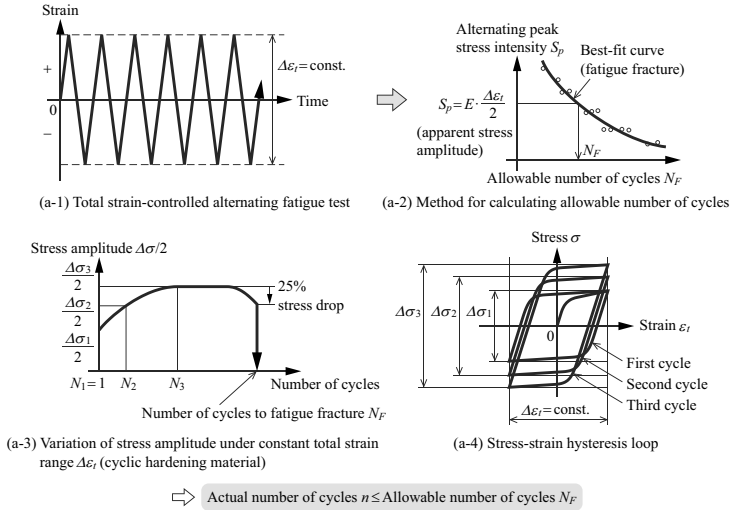
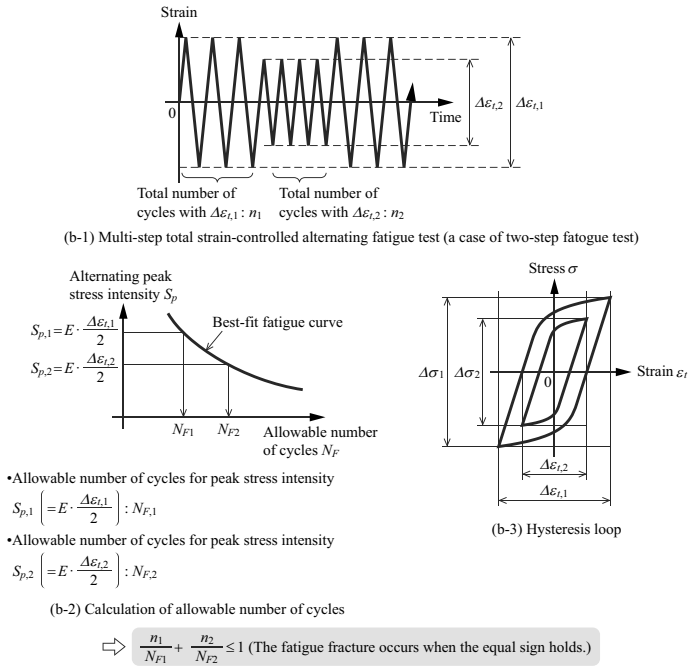


Fig. 5.22 Stress amplitude S_N' to be allowable under the condition of maximum effect of mean stress

and class 2 piping are allowed to be designed by applying the Design by Analysis approach. As is clear from the discussions so far, The Design by Analysis approach is applied to components critical for safety, such as class 1 components, because they require a very high level of quality, as mentioned above. However, for class 1 components, such as pumps, that have long and widely been used and are standardized by operating pressure, nominal diameter, and component type, the Design by Formula approach is considered to ensure adequate reliability, taking into account operating conditions. Class 2 components, which are those directly connecting to the reactor coolant pressure boundary, and class 3 components, which are those located outside the class 2 components (or, indirectly connecting to the reactor coolant pressure boundary), are considered to be properly designed by the Design by Formula approach, considering the required level of quality and the operating conditions. Although metallic reactor containment vessels, as classified as class MC vessels, is designed in accordance with the Design by Formula approach, its portions that are subjected to severe thermal loads are allowed to design according to the Design by Analysis approach. In the United States, the ASME Boiler and Pressure Vessel Code Section III Division 1 [2] is used as the code for nuclear reactor facilities, especially for light water type reactor facilities. The basic design approaches in the ASME Code are the same as Table 5.2. For Class 2 vessels, the Design by Analysis approach is available as an alternative design approach. The Design by Analysis approach as a



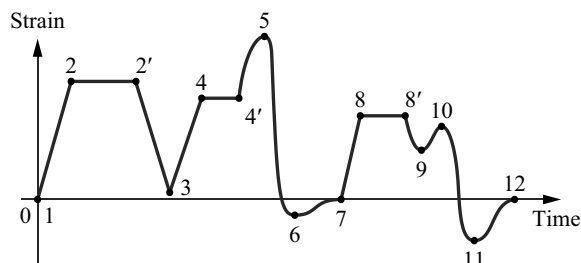
(a) Typical fatigue test and fatigue damage evaluation (for fatigue fracture prevention)



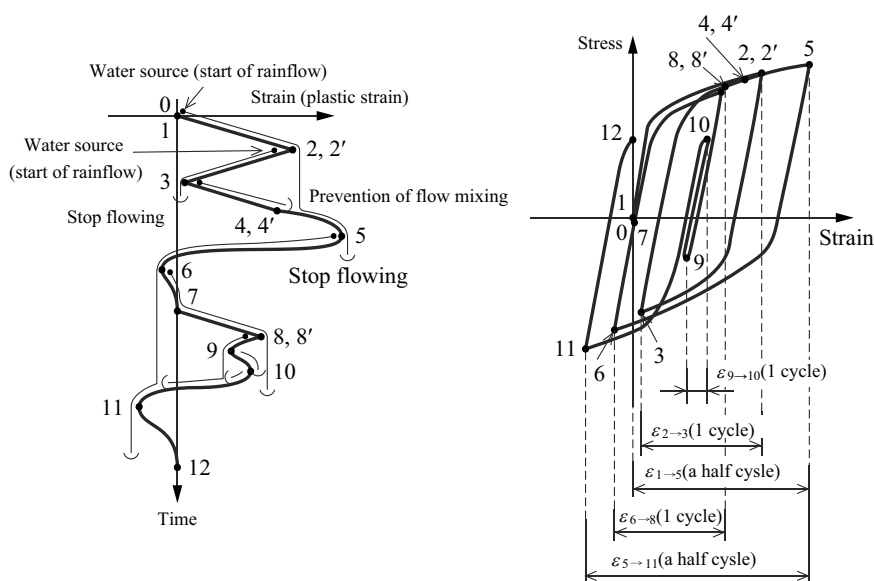
(b) Multi-step fatigue test and fatigue damage evaluation

[Note] Design fatigue curves are used in rules for evaluating fatigue damage in design codes. The design fatigue curves are generated incorporating design factors (for number of cycles, 20, while for stress amplitude, 2) and mean stress effect from the best-fit fatigue fracture curve.

Fig. 5.23 Fatigue tests with constant strain range and two-step fatigue tests



(a) Strain-time history expected to be experienced in a real plant component



(b) Application of rainflow method

Fig. 5.24 Rainflow method for counting irregular wave forms

general approach applied to vessels is described in Sect. 5.6. Stress limits applied to Class 1 vessels are described in detail in Sect. 5.9.2 to show how the approach is applied. Table 5.7 in Sect. 5.4.3 briefly describes the Design by Analysis approach that can be applied to Class 2 components as an alternative approach in the United States.

First, it should be noted that how the Design by Analysis and Design by Formula approaches are applied differs from one component to another because component configuration and loading conditions are specific to each component. A piping system is described in Sect. 5.7 as a typical example different from a vessel.

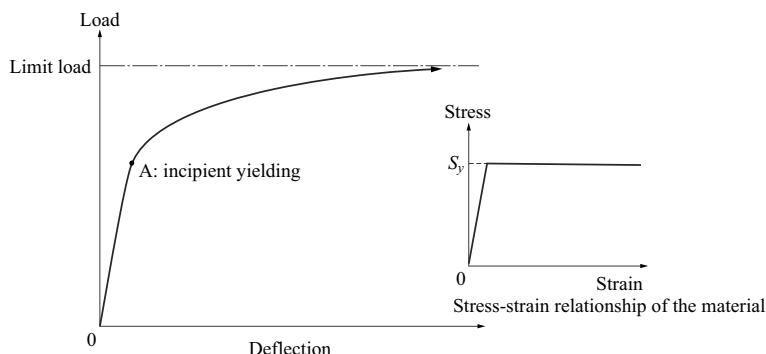
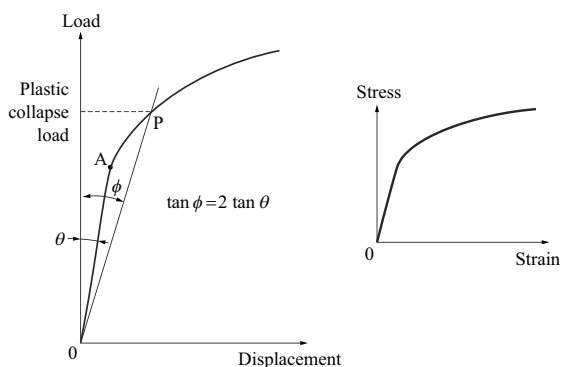


Fig. 5.25 Definition of limit load

Fig. 5.26 Plastic collapse load by application of the twice-elastic-slope method



Second, it must be avoided to use the design stress intensity for the Design by Analysis approach as the allowable stress and use only the formulas of the Design by Formula approach for wall thickness calculation and stress evaluation.

As described above, the design approach differs from component to component. Also, the design factor differs from approach to approach. Each other is closely and complicatedly interrelated. In Sect. 5.2 and later, the design factor and other items are described individually. In parallel, how they differ with the design approach and are interrelated is described.

[7] Stress intensity in a three-dimensional stress field

Configurations of most structures are three dimensional and therefore the stress in the structure is three dimensional. In Eq. (5.1), R represents strength such as tensile strength and yield strength. Data on material strength are obtained from a tensile test. In a tensile test, a specimen is stretched by an axial load. Therefore, the tensile strength and the yield stress are values for one-dimensional loading and scalar quantities. Generally, the material strength constituting the strength R is a scalar quantity. Then, the left-hand side of Eq. (5.1) is three dimensional and the right-hand side is one

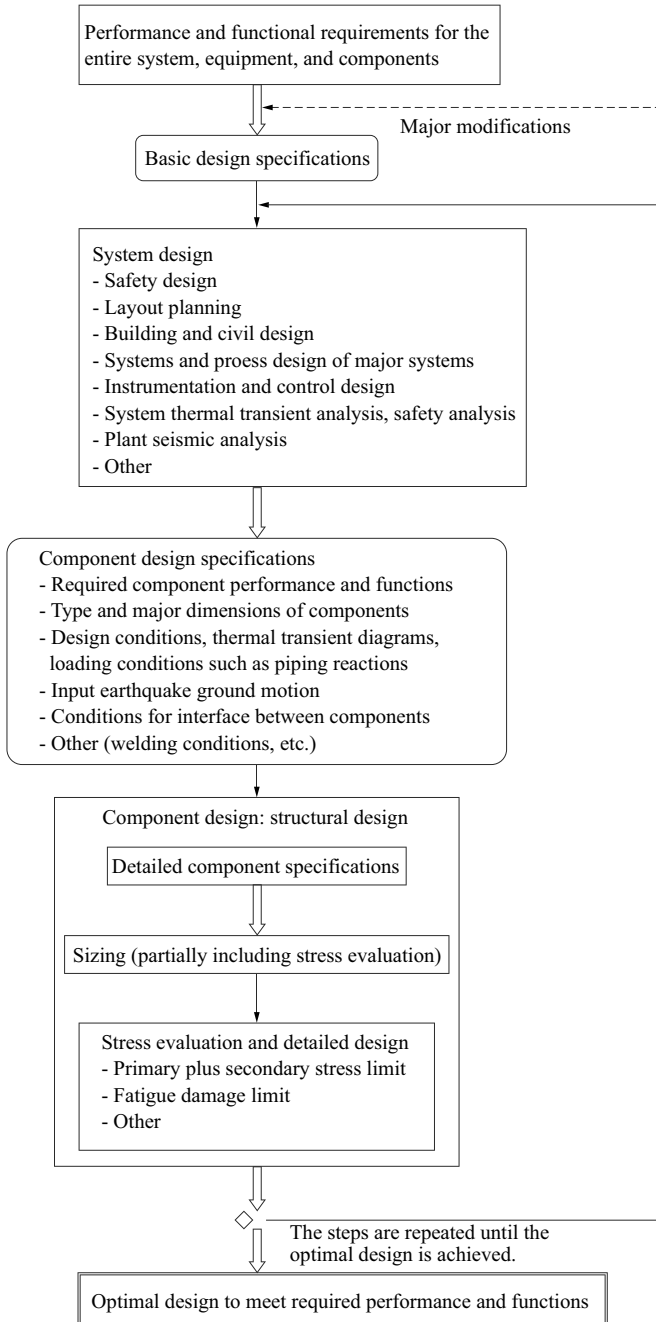


Fig. 5.27 Flow of structural and other design processes according to Design by Analysis approach

Table 5.1 Difference in vessel wall thickness due to application of different design approaches

	"Design by Formula" approach	"Design by Analysis" approach
Assumptions	•Loading conditions and service conditions: Very simplified conditions as follows; Internal pressure only acts on the vessel without any other load such as a thermal load. Only a normal startup and shutdown is assumed without any abnormal event such as a scram. •Design conditions for wall thickness calculation: - Maximum allowable working temperature (or design temperature) T_D; 350 °C - Maximum allowable working pressure (or design pressure) P_D; 17.0 MPa •Vessel configurations: - Cylindrical shell vessel without any structural discontinuity and 4.0 m in diameter •Vessel material: - Mn-Mo steel in use for nuclear reactor pressure vessels (SQV2A in JIS G 3120, and A533B in ASTM)	
Allowable stress	•Assumptions: - Design tensile strength $S_u = 490$ MPa and design yield strength $S_y = 285$ MPa, referring to JSME Codes for Nuclear Power Generation Facilities, Rules on Materials for Nuclear Facilities. - Note that at an elevated temperature above the normal temperature, the S_u values given in the material code is usually required to be multiplied by 1.1 before calculating the allowable stress (the multiplier 1.1, however, is not required in some codes, for example JIS, because this multiplier is incorporated into S_y values tabulated in these codes). •The allowable stress should be calculated according to the description provided in the material code, but here, as a simplified, it is calculated from values of design tensile and yield strength at the relevant temperature of 350 °C.	
	•Allowable tensile stress S: $S = \min(1.1S_u/3.5, 0.625S_y)$ $= \min(154, 178) = 154$ MPa	•Design stress intensity S_m $S_m = \min(1.1S_u/3.0, 2S_y/3)$ $= \min(180, 190) = 180$ MPa
Equation for calculating wall thickness t	•The basic stress is the maximum stress, and in this case, the circumferential (or hoop) stress of the cylindrical shell subjected to internal pressure. •Wall thickness t: $t = \frac{P_D \times D_i}{2S \times \eta - 1.2P_D}$ where D_i: inner diameter of the shell η: ligament efficiency	•Without any primary bending stress and local primary membrane stress, and the design conditions only are required to evaluate as the service conditions, it is enough to meet the requirement of the primary general membrane stress intensity P_m limit for the design condition: $P_m \leq S_m$ •The stress intensity P is defined as twice the maximum shear stress, and then the maximum difference between the principal stresses σ_1, σ_2 and σ_3 as follows, $P = \max(\sigma_1 - \sigma_2 , \sigma_2 - \sigma_3 , \sigma_3 - \sigma_1)$, $\sigma_1 \geq \sigma_2 \geq \sigma_3$ •In a case of thin-walled cylindrical shell subjected to internal pressure only, assuming no shear stress, $\sigma_1 = \sigma_\theta = P_D \times D/2t$ $\sigma_2 = \sigma_z = P_D \times D/4t$ $\sigma_3 = \sigma_r = -P_D$ Therefore, the wall thickness should meet the following inequality; $P_m = \sigma_3 - \sigma_1 = P_D + P_D \times D/2t \leq S_m$ Accordingly, $t \geq \frac{P_D \times D}{2S_m - 2P_D}$

(continued)

Table 5.1 (continued)

	“Design by Formula” approach	“Design by Analysis” approach
Calculation of t	•Let the inner diameter be 4 000 mm, $t = \frac{17.0 \times 4\,000}{2 \times 154 - 1.2 \times 17.0}$ $= 236 \text{ mm}$	•Let the diameter D be 4 000 mm, the minimum wall thickness is derived, $t = \frac{P_D \times D}{2S_m - 2P_D}$ $= \frac{17.0 \times 4\,000}{2 \times 180 - 2 \times 17.0} = 208 \text{ mm}$
Remarks	The Design by Analysis approach enables to reduce the wall thickness. However, as shown above, the wall thickness reduction causes the higher stress, and therefore, the Design by Analysis approach is allowed to apply as a design approach by imposing several more stringent requirements to assure the same level of safety as the Design by Formula approach. The detail is mentioned in Section 5-3-2.	

Table 5.2 Application of different design approaches, depending on class and type of component and supporting structure [1]

Class	Vessel	Pipe	Pump	Valve	Supporting structure
Class 1 components	Design by Analysis	Design by Analysis	Design by Formula	Design by Formula (based on reference pressure and temperature). Partially, simplified design by Analysis	Design by Analysis incorporating Architectural Institute of Japan's "Design Codes for Steel Structures" (fatigue evaluation is not required)
Class MC components	Basically, design by Formula				Rules for Class 1 supporting structures are applied.
Core supporting structures					Design by Analysis
Class 2 components	Design by Formula	Design by Formula	Design by Formula	Design by Formula (based on reference pressure and temperature)	Rules for Class 1 supporting structures are applied to primary stress for service conditions A and B
Class 3 components	Design by Formula	Design by Formula		Rules for Class 2 valves are applied.	Rules for Class 2 supporting structures are applied.
Class 4 piping systems		Rules for wall thickness, weld design and others			

dimensional. To formulate Eq. (5.1), the three-dimensional stress field for the load effect L must be converted to a one-dimensional stress field. Such typical cases are a yielding criterion and fracture criterion. It is not always appropriate to formulate such criterion by means of simply adopting the maximum stress component and then comparing it to the material strength or resistance. The three-dimensional effect should be adequately taken into consideration for formulating Eq. (5.1). To meet

this requirement, the concept developed to compare a three-dimensional stress field with one-dimensional data is the equivalent stress. As described in Chap. 3, various proposals have been made for the equivalent stress. The important point is that the definition of the equivalent stress used in Eq. (5.1) differs from one design approach to another.

In the Design by Formula approach, the maximum stress criterion is used because it has long been used and simplifies calculation. This criterion uses the maximum (particularly, normal stress) of three-dimensional stress components as L and compares it with the allowable stress R/DF .

In the Design by Analysis approach, stress intensity (should not be confused with the stress intensity factor used in fracture mechanics) by the maximum shear stress criterion (so-called Tresca's criterion) is used as the equivalent stress. The reason for this is that the maximum shear stress criterion is far more reasonable than the maximum stress criterion in predicting the yield, ductile fracture and fatigue failure of ductile metals. The criterion ensures the adequate consistency of the stress conditions (particularly, the direction of the stress vector) expected in components with test data. More importantly, it always provides conservative predictions of test data with an appropriate margin. The shearing strain energy criterion or the distortion energy. This criterion is called Mises criterion, because Richard von Mises proposed is good for the equivalent stress. However, the maximum shear stress criterion provides slightly more conservative predictions (by up to about 15%) than the shearing strain energy criterion, and was easy to use in the analysis method (not the finite element method (FEM) but the structural mechanics as introduced in Chap. 2) used in the 1960s. For these reasons, the maximum shear stress criterion was adopted, taking into account the directionality applied to cases where, in fatigue damage evaluation, the sign of shear stress changes in one cycle. However, today, the FEM is the dominant method and the shearing strain energy criterion is easier to use for FEM analysis. For this reason, the use of the strain energy criterion is being considered.

5.2 Failure Modes

As described in [2] in Sect. 5.1, there are five potential failure modes of light water reactor components. The design by analysis approach aims to explicitly prevent all of them. The failure modes that may really occur in the light water reactor components are not limited to these environmental-condition-independent modes, but shall include environmental-condition-dependent modes such as stress corrosion cracking and environmental fatigue. However, experience shows that the following three failure modes are particularly important.

- Ductile fracture (or the burst of a component resulting from ductile fracture), plastic collapse, and gross distortion from a single (or monotonic) load application
- Progressive distortion from cyclic application
- Crack initiation from fatigue damage

As described in [2] in Sect. 5.1, the stress causing these failures is not necessarily the stress (called the total stress) obtained by stress calculation. The nature of the stress varies with the type of load that causes the failure. Also, its effect on failure varies with the nature of the stress distribution. For example, a membrane stress, defined as the component of normal stress that is uniformly distributed, causes necking if the stress is generated by an external load, resulting in unstable and localized plastic deformation and subsequently in ductile fracture and then rupture. On the other hand, a bending stress generated by a bending moment does not cause necking, but a stable plastic deformation until a bending crack forms on the tensile surface where a maximum tensile stress is developed. In some cases of cylindrical pipe subject to the bending moment, a local buckling may occur in a compressive stress area, or a unique deformation called the “Brazier effect” may be developed.

A burst requires that the external load continue to be applied to the structure after large plastic deformation and stress that causes further deformation be present in the structure. Stress is required to cause the crack to grow. Deformation-controlled stress such as thermal stress occurs only when deformation is constrained as in a bimetal strip. Due to this nature, the stress decreases when residual plastic deformation occurs or the boundary constraint conditions are relaxed. As a result, the deformation-controlled stress does not cause neither a burst nor gross distortion. Its single load application does not lead to structural failure.

Thus, the failure mode depends on the nature of the stress (precisely, stress component). Table 5.3 shows the relationship between the failure modes and the types of stress. The failure modes that may really occur in the light water reactor components are not limited to these modes, but shall include environmental condition dependent modes such as stress corrosion cracking and environmental fatigue. The types of stress are first described below. Then, what type of stress causes what mode of failure is described based on the characteristics of the failure mode.

5.2.1 *Types of Stress*

The types of stress are described in Sect. 3.3.1. Here, those are described in detail from the standpoint of structural codes or formulating Eq. (5.1). Stress is divided into two main types in terms of the load to induce stress: load-controlled stress, that is resulted from an application of external load, and whose magnitude is not reduced as a result of displacement, and deformation-controlled stress, which changes according to deformation as when deformation is constrained.

There is another classification of stress. The stress that is developed by an external load and is necessary to satisfy the law of equilibrium of the loads is called primary stress (precisely, primary stress component). The definition is given in NB-3213.8 of Ref. [2], for instance. To put it simply, load-controlled stress can be considered to be a primary stress. From a viewpoint of failure prevention, a clear and straightforward definition of stress classification is needed to prevent any confusion about the relation between failure modes and stress components. The stress component to cause the

Table 5.3 Failure modes, important types of stress, and related material strength

Failure mode		Type of stress	Related material strength
1. Burst, plastic collapse, etc.	Burst, ductile fracture, plastic collapse and gross distortion	General primary membrane stress plus bending stress (even by only one application of loading)	Tensile strength
	Local but excessive plastic deformation	Local primary membrane stress plus bending stress (even by only one application of loading)	Yield stress
2. Loss of function due to progressive distortion		Range of primary plus secondary stresses due to cyclic loading	- Yield stress - Local strain (reduction of area)
3. Fatigue failure		Range of the total stress, including peak stress, due to cyclic loading	Fatigue strength
4. Buckling	Load-controlled buckling, such as column buckling and buckling under external pressure	Compressive primary stress (there are cases where buckling is caused by tensile stress, such as creases in the pressure vessel head) (even by only one application of loading)	Critical buckling stress, etc.
	Deformation-controlled buckling due to constraint of thermal expansion, etc.	Deformation-controlled compressive stress (even by only one application of loading)	Buckling strain
5. Brittle fracture		Limit on the minimum working temperature instead of stress (even by only one application of loading)	Fracture toughness

ductile fracture shall increase as the applied load increases, so as to satisfy the law of equilibrium of the loads, even though the deformation increases with increasing the load. Deformation-controlled stress is divided into a component (called secondary stress) that can cause distortion and thus ratcheting and a component (classified as peak stress) that causes only fatigue failure. The stress due to the self-constraint of a structure or the constraint from the adjacent member is defined as secondary stress. The definition is similarly given in NB-3213.9 in Ref. [2]. The stress obtained by subtracting primary and secondary stresses from the total stress is called peak stress, which appears at structural discontinuities such as a notch. This type of stress component does not cause any plastic deformation to such an extent as to cause ratchetting, i.e., does not cause any significant distortion. Other type of peak stress is a local thermal stress. The local thermal stress may be developed in a small hot spot in a vessel.

Due to its distribution characteristics, as shown in Fig. 5.4, primary stress can be divided into a membrane component (the average across the thickness) and a bending component, which are called membrane stress and bending stress, respectively. The stress distributions in the figure are those in a structurally smooth (or plain) region

remote from any discontinuity (called the general region). The bending component can be divided into a linear component and the remaining nonlinear component. The linear component is an equivalent linear component of the stress distribution. The equivalent bending moment component of the distribution obtained by subtracting the membrane component from the stress distribution is presented in the form of a linear stress distribution.

In American and Japanese structural codes, the following symbols are assigned to clearly differentiate types of stress intensity and stress component:

Primary stress intensity: P

General primary membrane stress intensity: P_m .

Local primary membrane stress intensity: P_L .

Primary bending stress intensity: P_b .

Secondary stress intensity: Q

Peak stress: F

(Note that the peak stress intensity that is also denoted as F is derived from a calculation of all primary, secondary and peak stress. The detailed definition is given in NB-3222.4(b) of Ref. [2]).

Figure 5.5 illustrates examples of stress classification into primary, secondary and peak stresses in a case of a pressure vessel. It should be noted here that the secondary stress may be divided into membrane and bending in principle. Such a classification of secondary stress, however, is not needed because the secondary bending stress component can be controlled by the same stress limits as the secondary membrane stress component [2].

In reality, the loads acting on components are not external loads or thermal loads only. Generally, they are applied simultaneously. Therefore, both primary and secondary stresses are generated, and a peak stress is also generated simultaneously at structural discontinuities.

Accordingly, the above-mentioned classification pertains to components of generated stress rather than stress itself.

5.2.2 Failure Modes

The basic concept of the failure modes is described in Sect. 3.2 in Chap. 3. This section identifies important points in formulating Eq. (5.1) and describes them in detail.

Among the failure modes shown in Table 5.3, brittle fracture is prevented by limiting the minimum working temperature, instead of the stress limit given by Eq. (5.1). The purpose of this is to ensure adequate fracture toughness to prevent an unstable fracture due to potential cracks. Consideration of the potential cracks is owing to the possibility that there may be a flaw whose size is below the detection

lower limit of nondestructive devices, and a rare possibility of escaping defect. This is described in detail in Chap. 11.

Failure, fracture and rupture should be defined before a discussion of the failure modes. The following definitions may be used in strength of materials [3].

- Failure: a phenomenon in which plastic deformation occurs in a material

Plastic deformation is shear deformation on a slip plane due to the movement of dislocations, which is called slip deformation, and therefore is irreversible. Slip deformation is caused by shear stress and therefore involves no change in volume.

- Fracture: a phenomenon involving large cracks.
- Rupture: a phenomenon involving tearing apart.

The relationship between these definitions and their corresponding illustrations of the specimen in a tensile test can be given as in Fig. 5.6.

The definition of failure needs explanation. Failure in this book is defined as the state in which a structure or its structural element cannot perform the function (pressure resistance (or pressure-retaining), leak-tightness, etc.) required by the structural codes. This type of state often results from the plastic deformation of the material. The definition of failure in this book is different from that in Ref. [3], but, in terms of results, means the same as that in the reference.

[1] Burst, plastic collapse and gross distortion

We consider a cylindrical vessel subjected to an internal pressure as shown in Fig. 5.7. The circumferential stress σ_θ in the cylindrical shell of this thin vessel, with a diameter D and a wall thickness t , subjected to an internal pressure p is given by the following equation from the balance between the internal pressure and the stress as shown in Chap. 2:

$$\sigma_\theta = \frac{D}{2t} p \quad (5.2)$$

This relationship remains true even if yielding occurs as the internal pressure increases. With a further increase in pressure, gross plastic deformation (gross distortion) occurs, resulting in necking and eventually causing a “burst”. This is because even if the vessel is distorted, the stress is uniquely given by Eq. (5.2) and not relaxed by the distortion, and the increasing pressure increases the deformation. When the deformation instability condition represented by tensile strength is reached, the unbounded deformation may occur without an increase in pressure. Alternatively, if cracks are initiated and the instability condition for the cracks is reached due to a further increase in pressure, an unstable fracture occurs and therefore, the cracks grow fast, resulting in a burst.

Thus, burst and gross distortion occur when stress occurs to balance the load (or to satisfy the law of equilibrium of loads) and does not reduce the magnitude due to deformation by the load. In other words, they occur when a load is applied that

causes primary stress. Therefore, primary stress is limited to prevent burst and gross distortion.

Even under a load that causes primary stress, unlike a cylindrical shell with no structural discontinuities, in the tapered region where the wall thickness changes and structural discontinuities such as the knuckle between the cylindrical shell and the head, the stress is redistributed according to the plastic deformation after yielding because of the constraint from the surrounding region. Excessive plastic deformation at structural discontinuities is considered to be a failure in the structural codes on the safe side. Therefore, this stress is classified as a local primary membrane stress from the perspective of failure prevention, although it is a secondary stress in nature.

In terms of the failure mode that can be caused by primary membrane plus bending stress, for example, the primary membrane plus bending stress developed in the general region, which is far from a structural discontinuity, yielding occurs as the load is increased and gross distortion occurs with a further increase in load. Even if necking does not occur by further increasing the load (for instance, in a structural element subjected to pure bending, no necking occurs), collapse (or plastic collapse) involving significant plastic deformation occurs, resulting in initiation of a crack on the tensile surface, and finally, the load-carrying capacity may be lost. The detail is mentioned in Item [2] in Sect. 5.6.1. It should be noted here that if a membrane stress component is dominant (or, an axial or tensile load to induce the membrane stress component is much larger than a bending moment to cause the bending stress component), necking starts, and then unstable deformation is developed, finally resulting in ductile fracture. On the other hand, if the bending stress component is dominant, the necking does not occur even if the deformation increases, but a plastic collapse occurs, finally leading to ductile fracture due to growth of a crack initiated on the tensile surface (called “bending cracking”). It should be noted here that under combined loading of tensile load and bending moment, there is a possibility of necking even if the bending stress component is dominant. The reason is because the cross section is reducing as the tensile load increases.

For local primary membrane plus bending stress, if yielding occurs, excessive plastic deformation occurs locally due to the constraint by the neighboring region.

From the above discussion, as shown in Table 5.3, limits imposed on the general primary membrane plus bending stress are to prevent failures of burst, ductile fracture, and plastic collapse. The material strength involved is tensile strength. Gross distortion is also prevented by limiting general primary membrane plus bending stress. Excessive deformation at structural discontinuities can be prevented by limiting local primary membrane plus bending stress. The yield strength is mainly involved in gross distortion and excessive deformation.

As previously mentioned, the stress with regard to the primary stress actually means the stress vector in three-dimensional stress field. Conversion of the stress vector to one-dimensional scalar quantity, i.e., the equivalent stress is needed for comparing to stress limits, “Stress intensity” is used as the equivalent stress in many codes and standards. Therefore, it is correct to use the term the “general primary membrane stress intensity”. In this section, however, the “general primary membrane stress” is used as its abbreviation. It should be noted here that in such cases as

thermal stress ratchetting limit formula, the stress means the stress component itself but not stress intensity, because these formulae are derived for the stress component concerned.

Burst is discussed in a little more detail below. Burst is likely to occur in a pressure testing. Of course, it can occur after a certain period of operation. If pressure-retaining components are manufactured as designed, the significance of pressure-testing halves (one of the purposes of pressure testing is the relaxation of residual stress by plastic deformation). However, possible errors, for example in material selection, design, welding, and examination, should be considered in the construction stage. The most important purpose of pressure testing is to warrant that pressure-retaining components have been constructed without errors, or that they have adequate pressure resistance. Burst is the failure mode that can occur when such errors are present. In terms of the allowable stress for preventing burst, tensile strength is used as material strength and a design factor is added to it. The design factor is determined empirically but not by design analysis, which does not consider errors. At the moment, it has to be empirically determined what errors can exist and what the design factor should be for the stress that causes burst. For example, if the number of undetected defects decreases with an improvement in welding quality and examination performance due to technological advances, the design factor can be reduced. As a result, the pressure-testing pressure can be reduced, leading to an improvement in working pressure. From this historical background, in the ASME Boiler and Pressure Vessel Code, the design factor for tensile strength was determined as shown in Table 5.4 in Sect. 5.3.

The design factor was called the safety factor in the twentieth century. However, the term “safety factor” can lead to a misunderstanding that safety is lower with a lower safety factor. Therefore, in the twenty-first century, the term was displaced by “design factor (or design margin).”

Table 5.4 Changes in design factor on the (ultimate) tensile strength [5]
The design factor was revised as follows in the ASME Boiler and Pressure Vessel Code.

Item	Revision year	Issued or revised design factor
Allowable stress (allowable tensile stress) in “Design by Formula”	Issued in 1925 as Section VIII	5
	1942	Revised from 5 to 4
	1999	Revised from 4 to 3.5
Allowable stress (design stress intensity) in “Design by Analysis”	Issued as Section III in 1963 Issued as Section VIII, Division 2 in 1968	3

- [Notes]
- (1) In terms of the yield stress, the design factor on the design stress intensity was revised from 5/8 to 2/3 in 1979.
 - (2) In terms of the design stress intensity, the design factor on the tensile strength has recently been revised in Section VIII, Division 2.

[2] Functional loss by progressive distortion

Even under a single load application, primary stress can cause plastic deformation across the wall thickness as the load increases. When combined with primary stress, even non-primary stress can cause yielding in a manner to complement each other in one cycle of loading and unloading. For example, tensile yielding can occur on the outer surface of a vessel in the loading process and on the inner surface in the unloading process due to the residual strain from the loading process. Through one of loading cycle, tensile plastic strain can occur across the thickness. You should be reminded of what is described in Sect. 3.4.2 on the case of two bars, which are simple structural elements. Under cyclic loading, the plastic deformation is accumulated across the thickness and eventually the required function (for example, providing an adequate gap between the heat transfer tubes to ensure coolability) may not be performed. Against such loss of function, measures should be taken for ensuring the structural integrity. Therefore, loss of function due to excessive deformation should be treated as a failure. This failure may be called functional loss due to progressive distortion.

A typical example of this in structures in service is thermal ratcheting, which can occur when a temperature gradient occurs due to heating and cooling across the thickness of a cylinder constantly subjected to an internal pressure, and the thermal stress due to the temperature gradient is cyclically applied. Thermal stress occurs when a thermal load is applied, and is caused for instance, due to a non-uniform temperature distribution in the structure or the difference in the coefficient of thermal expansion in the structure.

Suppose the structure is constrained (to be exact, self-constrained) to deform and subjected to a thermal load. As previously described, the stress that is developed to meet the strain distribution conditions compatible with the constraint is called secondary stress. As a result of stressing, the structure may be distorted. Therefore, it is called self-balancing stress because it is developed to balance the constraint without an external load.

It should be noted that thermal stress is not necessarily a secondary stress and can be the peak stress described previously at structural discontinuities. In many cases, both secondary and peak stresses are present. In short, the membrane plus bending component of deformation-controlled stress is a secondary stress and the remaining component at a structural discontinuity is a peak stress. The structure can be distorted by general thermal stress as a secondary stress but is not significantly distorted by local thermal stress as a peak stress. Thermal stress with the nature of peak stress can occur in regions other than structural discontinuities. A typical one is the thermal stress that occurs as a hot spot. Deformation-controlled stress as a peak stress occurs in a limited area, and therefore, locally causes a plastic deformation. This means that the peak stress does not cause any plastic deformation across the thickness through a cycle of loading and unloading. As a significant distortion is not developed, the peak stress does not cause the failure of a loss of functions due to progressive distortion.

It should be mentioned again here that stress classification into primary, secondary and peak stress components has been incorporated in major structural codes and

standards owing to simple, clear and one-to-one correspondence of stress components with failure modes, instead of classification into load-controlled and deformation-controlled (or displacement-controlled) stress component.

From the above discussion, the following limits, as well as limits on deformation, are applied to prevent the failure of this mode and to satisfy several assumptions to be made in formulating expressions provided in codes and standards. The limits on deformation as usually given as numerical values of distortion depend on specific operating conditions of the plant concerned. This means that these limits should be given in design specifications of each plant, but not prescribed in generic codes and standards.

The basic philosophy of preventing incremental collapse or ratchet failure is to prevent excessive plastic deformation. The structural codes and standards require an elastic response of a structure to cyclic loading. More specifically, the structure is allowed to slightly yield for the first few cycles. The subsequent structural response is required to be elastic due to the effect of residual stress. The phenomenon in which after yielding, the deformation remains in the elastic range due to the residual stress is called “shakedown.” As a rule, the range of primary plus secondary stress intensity is limited to $2S_y$ (S_y is the design yield point (or yield strength)). The material strength involved in preventing ratchet failure is the yield stress. However, in the current structural codes, $2S_y$ is displaced by $3S_m$ (S_m is the design stress intensity). The reason for this is described in Sect. 5.6.2.

For thermal ratcheting, rules for raising the limits instead of the above-mentioned limits are set based on the analysis by Bree et al., which is described in Sect. 5.6.2.

As a rule in the current structural codes, fatigue damage evaluation to prevent fatigue failure (to be discussed below) is based on elastic analysis, although rules are available for cases where the structural response enters the plastic regime. The above rule is set to not only meet the shakedown criterion but also to warrant the validity of elastic analysis for fatigue evaluation.

[3] Fatigue failure

In the construction stage, all detectable defects must be removed for guaranteeing the defect-free design. However, it is unrealistic to completely remove surface flaws such as scratches and weld flaws because nondestructive examination devices have their lower limit for detecting a flaw. When even a specimen with a mirror finish is subjected to cyclic loading, a crack is nucleated at a certain cycle on a surface as a result of repeated plastic deformation, as described in Chap. 3. If the specimen has pre-existing small scratch and flaw on the surface, the crack nucleates earlier than for the specimen with mirror finish. If a crack exceeds a certain size, it begins to grow and reaches the size called the crack initiation size. (There are different definitions of crack initiation sizes, and the size depends on the definition.) With further cyclic loading, the cracks (called fatigue cracks) grow to the point where the test must be stopped due to a decrease in load-carrying capacity resulting from crack growth. In vessels and piping in service, for example, fatigue cracks penetrate the wall of the vessel or piping. The penetration causes leakage of fluid contained in the vessel or piping.

Crack growth and penetration can be evaluated to a reasonable extent with the aid of fracture mechanics. However, there is no established reasonable method for evaluating crack initiation, including the definition of the size of flaw that initiates cracking. The ratio of the number of cycles to crack initiation to the number of cycles to failure is different between so-called low-cycle fatigue failure, which occurs at low cycles of 10^3 to 10^4 at a relatively high stress amplitude, and high-cycle fatigue failure, which occurs at cycles higher than 10^5 at a low stress amplitude. Crack initiation occurs earlier at low-cycle fatigue.

As discussed above, fatigue failure (precisely speaking, fatigue fracture) can be characterized by crack initiation and subsequent crack growth in each cycle. Crack initiation and growth is caused not only by an external load such as pressure and other mechanical loads, but also a deformation-induced load such as a thermal load. That is, it is caused by not only load-controlled stress but also deformation-controlled stress.

The following are typical causes of fatigue failure:

- A change in pressure associated with normal startup and shutdown and a change in reactor power
- A change in temperature associated with startup and shutdown, a change in reactor power, and emergency shutdown; thermal vibration such as thermal striping due to an alternating flow of hot and cold fluids observable at points where piping merge, such as at T-joints. Fatigue due to these causes is called thermal fatigue.
- Earthquakes
- Flow-induced vibration: vibration of a cylinder due to an alternating shedding of vortices called Karman vortices or hydro-elastic unstable vibration
- High-frequency pressure waves generated by cavitation in pumps and valves.

Traditionally, in the Design by Formula approach, fatigue evaluation is not performed. This is because in boilers and pressure vessels at general plants other than nuclear power plants, it was recognized empirically that failure rarely occurred with a design factor of 5 or 4 (currently, 3.5) for the design tensile strength as the allowable stress although fatigue evaluation was not performed. Fatigue evaluation, when performed, was primarily based on so-called an endurance limit (the lower limit for fatigue failure of carbon steel and so on, which means that failure does not occur at a stress amplitude below this limit). This approach is called endurance limit design. However, in the Design by Analysis approach, in exchange for a reduction in design factor from 4 to 3, evaluation of all of the potential failure modes is required in the design codes and further, aiming at controlling cracks, more stringent requirements are placed in fabrication, examination and testing codes and so on. Importance of fatigue evaluation was recognized because of higher allowable stresses. The number of load (stress) cycles for events relating to thermal fatigue, earthquakes, etc. postulated during the design life rarely exceeded 10^5 . Therefore, low-cycle fatigue evaluation became important in fatigue evaluation. Low-cycle fatigue was not familiar in traditional mechanical design, and required the development of a new evaluation method. The concept of representing the level of damage with the usage factor given by the ratio of the number of cycles n/N_f was proposed (n is the number of design load

cycles for stress amplitude at the event under consideration and N_f is the number of the cycles to failure under cyclic loading at the stress amplitude). If several operating patterns are involved, the cumulative usage factor, the linear sum of partial usage factors $n_i/N_{f,i}$ for each cycle pattern, is used to represent the level of damage. The concept of usage factor is based on the assumption that fatigue damage per one load cycle is represented as $1/N_{f,i}$. The Palmgren–Miner linear damage relationship was incorporated into ASME Boiler and Pressure Vessel Code Section III in 1963.

[4] Buckling

As shown in Fig. 2.28 in Chap. 2, a slender column subjected to an axial compressive load buckles when the load reaches a critical value. This type of buckling is called column buckling. The criterion for this type of buckling is described in Sect 2.6 in Chap. 2. Buckling under complex loading and buckling of complex structural geometries are described in Sect 3.4.4 in Chap. 3. Being pressure-retaining components subjected to a very high pressure, light water reactor components are heavy walled but require the consideration of buckling.

Do not assume that buckling occurs only in thin structures. One example is the reactor containment vessel. For example, some PWR steel containment vessels have a diameter as large as 40 m. The vessel wall is as thick as 5 cm. It looks very thick, but is thin compared to the very large diameter. The design pressure (to be exact, the maximum allowable working pressure in Japanese codes and standards) is only 0.3 MPa (3 atm in the old-fashioned way). Consequently, the possibility of buckling must be considered if the external pressure of the vessel is 0.01 MPa (=100 hPa) higher than the internal pressure. The principle of the rules for preventing buckling of vessels subjected to an external pressure is described later.

Generally, the unstable condition (particularly, statically) of structures and members is defined as the condition in which small load increments, etc. cause large deformation (particularly, unbounded deformation). Buckling is a typical example of this unstable condition. Another typical example is deformation instability observable in a tensile test. The latter is distinguished as plastic tensile instability from the former (i.e., buckling). The definition of plastic tensile instability is given in NB-3213.26 in Ref. [2].

5.3 Design Factor

5.3.1 Concept of the Design Factor

The allowable range of load and stress in the structural codes and standards is given by Eq. (5.1). Normally, the “minimum design value” taking into account scatter of test data is defined as the strength R . Due to uncertainties caused by design errors, fabrication mistakes, etc., in the stages of material production, design, fabrication, examination and testing, and further operation, the allowable range of the load L is

limited to below R/DF so that the probability of structural failure is kept below the allowable level. In the current codes, the design factor in this equation is set to a value that ensures, based on experience or rather field records, that the failure level is in the allowable range.

The following are a few examples of the uncertainties other than those caused by human errors such as design errors and fabrication mistakes:

- Materials always involve uncertainties (as mentioned above, scattering in test data are considered by setting the value of material strength R in Eq. (5.1) as a lower confidence limit of the test data, for example.).
- In material acceptance examination, possible harmful defects are detected and removed, including material producing processes. However, there are possibilities of defects (with a variation in size and depth) smaller than detection sensitivity limits and of escaping the detection.
- Uncertainties in manufacturing dimensions and structural configuration (variations within dimensional tolerances inevitably occur).
- Uncertainties in installation.
- Harmful defects are detected by welding examination and removed (removal of detectable defects is required by the codes and standards). However, there are possibilities of defects (with a variation in size and depth) smaller than detection limits and of escaping the detection.
- Uncertainties in microscopic stress distribution (stress in a polycrystal is not uniform due to the difference in resistance to intergranular and transgranular deformation).
- Uncertainties in applied loads.
- Uncertainties in modeling the structure for stress calculation as well as thermal transient analysis.

In the design codes, formulas were developed, with limits on the allowable range of uncertainties in manufacturing tolerance and configuration, under conservative conditions (for buckling, for example, under conditions that cause the worst initial imperfections, with limits on the allowable range of imperfections). However, as is discussed on a historical background of design factor in Item [1] in Sect. 5.3.2, many explosion accidents such as that at the steam boat Sultana in 1865, at which about 1500 people were killed. It became recognized that only adoption of a minimum design value in order to take into consideration the scatter of test data was not enough to prevent those explosions, and therefore it became essential to introduce a safety factor (now called the design factor to avoid a misunderstanding) for considering the above-mentioned sources of uncertainties.

In the current structural codes, allowable stresses to prevent failure due to primary stress, such as ductile fracture and plastic collapse, were determined as follows after a long effort to reduce the safety factor.

[1] Allowable tensile stress S used in the Design by Formula approach**(1) Ferrite materials (carbon steel and low-alloy steel)**

The least of the following:

- $1/3.5$ of the minimum tensile strength $S_u (RT)$ at room temperature RT
- $1/3.5$ of the design tensile strength $S_u (T)$ at temperature T for which the allowable stress is determined
- 0.625 times the minimum yield point (yield strength) $S_y (RT)$ at room temperature RT , where 0.625 is equal to $5/8$
- 0.625 times the design yield point (yield strength) $S_y (T)$ at temperature T

(2) Austenitic materials and high-nickel alloys

The least of the following:

- $1/3.5$ of the minimum tensile strength $S_u (RT)$ at room temperature RT
- $1/3.5$ of the design tensile strength $S_u (T)$ at a temperature T
- 0.625 times the minimum yield point (yield strength) $S_y (RT)$ at room temperature RT
- 0.9 times the design yield point (yield strength) $S_y (T)$ at temperature T

[2] Design stress intensity S_m used in the Design by Analysis approach**(1) Ferrite materials (carbon steel and low-alloy steel)**

The least of the following:

- $1/3$ of the minimum tensile strength $S_u (RT)$ at room temperature RT
- $1/3$ of the design tensile strength $S_u (T)$ at temperature T
- $2/3$ of the minimum yield point (yield strength) $S_y (RT)$ at room temperature RT
- $2/3$ of the design yield point (yield strength) $S_y (T)$ at temperature T

(2) Austenitic materials and high-nickel alloys

The least of the following:

- $1/3$ of the minimum tensile strength $S_u (RT)$ at room temperature RT
- $1/3$ of the design tensile strength $S_u (T)$ at temperature T
- $2/3$ of the minimum yield point (yield strength) $S_y (RT)$ at room temperature RT
- 0.9 times the design yield point (yield strength) $S_y (T)$ at temperature T

The design tensile strength S_u and the design yield point (or yield strength) S_y are determined as the minimums of data on tensile strength and yield stress, respectively, obtained from tensile tests at each temperature. The minimum value is the value determined as a design rule that corresponds to a failure probability of about 1%. The minimum tensile strength and the minimum yield point (yield strength) are the specified proof values unlike the design tensile strength and the design yield strength. The design factor on tensile strength was empirically determined as described as supplementary information on burst in Item [1] in Sect. 5.2.2. The design factor on other allowable stresses was also empirically determined.

The yield point (or strength) was incorporated as a factor to determine the allowable stress in order to guarantee the allowable stress range within the elastic design regime. A value of the design factor on yield point is thought to be determined from that on tensile strength as well as taking into account other factors such as limits on pressure testing. In determining the allowable stresses S and S_m , the reason why for austenitic stainless steels and high-nickel alloys, the constant 0.9 is multiplied by the design yield point is the large work-hardening capability of 2 or greater and a high ductility. Hence, they provide lower likelihood of causing a catastrophic situation than that for carbon steels, even though an applied stress has the possibility of exceeding the yield point.

However, with ever advancing technology, these uncertainties are decreasing. An effort to quantify them has been made vigorously. The frequency of causing human errors also have been decreasing because of actively and continuously conducting quality assurance activities. Reflecting the steady reductions in these uncertainties, the design factor has been revised and reduced every 30 to 50 years. The important point is that the reduced design factor is not at the cost of safety. The question is how much the design factor can be reduced, taking into account technological advances, while achieving the same level of safety as has been in the past. In this context, the traditionally used term “safety factor” can be misleading. To avoid a misunderstanding, the term was replaced by the term “design factor.”

5.3.2 Reduction of the Design Factor

[1] Design factor on the tensile strength to limit primary stress

For the allowable stress to limit primary stress to prevent failure such as ductile fracture and plastic collapse, the design factor on the tensile strength was reduced as shown in Table 5.4 with advances in technology.

According to Ref. [4], steam began to be widespread for industrial applications in the eighteenth century and power generation using steam produced by a boiler began in the latter half of the nineteenth century. Consequently, the frequency of boiler explosion accidents increased with the increased number of steam boilers used. Overpressure was the common cause of the accidents. An accident in Massachusetts in 1905 took lives of 58 people. Following this accident, the State of Massachusetts established “An Act Relating to the Operation and Inspection of Steam Boilers” in 1909. This act regulated the maximum operating pressure of boilers by the category of material used and established the procedures for certificate stamping. Subsequently, the State of Ohio established a similar act. There was increasing momentum for the development of boiler construction code uniform across the country among academic societies and associations in the United States. In September 1911, the Boiler Code Committee was established and started activities with members consisting of boiler manufacturers, operators, inspectors, and owners, and with particular attention to a balance of interests from its neutrality.

In 1915, the first issue (the 1914 version) of the boiler code was published. In the code, the maximum allowable working pressure p is given by

$$p = \frac{TS \times t \times E}{R \times FS} = \frac{t}{R} \times E \times S \quad (5.3)$$

where TS: ultimate tensile strength (the tensile strength in Sect. 5.3.1, as quoted from the original source),

t : minimum thickness of the shell plate,

E : efficiency of longitudinal joints or ligaments between tube holes,

R : inside radius,

FS: factor of safety (this term used to be used, but is replaced by the term “design factor” in the following),

$S = TS/FS$: allowable stress determined by the ultimate tensile strength.

The design factor (safety factor) $FS = 5$ was used in the code. Currently, several formulas to calculate the shell plate thickness are available for different thicknesses and are more complex than the above formula. The above formula is appropriate for the low working pressure of boilers with a thin wall thickness used in those days.

How the design factor for the ultimate tensile strength (or the tensile strength) was changed is described in Ref. [5]) (see Table 5.4). In summary, in 1925, a construction code for unfired pressure vessels was issued as the ASME Boiler and Pressure Vessel Code. In this code, the design factor was 5. The boiler code (Section I) and the code for unfired pressure vessels (Section VIII) were nearly identical, with some difference. For example, the applicable primary fluid for boilers was steam or water. The primary fluid contained unfired pressure vessels were a fluid that provides a severer operating environment, such as oil or chemicals.

In 1945, in Sections I and VIII, the design factor was reduced from 5 to 4 (as a code case in 1942 prior to this revision). The purpose of the reduction was to address the serious shortage of steel due to the demand during World War II. It is needless to say that improvements made in those days in design technology, material quality, and nondestructive examination made this possible.

In 1999, the factor was reduced from 4 to 3.5 in Section I, Section VIII Division 1 and Section III Classes 2, 3, MC, etc. due to primary stress. This reduction was made to meet the need for competing with the European codes. The decision to reduce the design factor was made, taking into account the fact that improvements had been made to the code as needed by actively incorporating technological advances made since the 1960s.

The reductions in design factor described above are related to the Design by Formula approach. In 1955, a larger reduction was requested by the oil industry to significantly increase the working pressure. The wall thickness of pressure vessels as thick as 20 cm (about 8 inches) was required for a design factor of 4. A study was started on how the design factor in Section VIII could be reduced to 3 to reduce

manufacturing costs without compromising the level of safety. As a result of the study, the following requirements were added to the code:

- Restrict the materials used.
- Add requirements for fracture toughness.
- Perform fatigue damage evaluation.
- Perform stress analysis under a wide range of loading conditions.
- Perform nondestructive examination of a wider area.

When these requirements were made, in the nuclear industry, the wall thickness of reactor vessels began to increase and reducing the wall thickness was an urgent issue. Further, nuclear reactors, in particular components important to safety such as a nuclear pressure vessel, are required to construct and operate at an extremely high level of quality, because of their serious hazard. As a result, the above requirements were added to allow a reduction in thickness without compromising the level of safety, and the design factor was reduced to 3. Thus, rules for the Design by Analysis approach were established. These rules for vessels for nuclear applications were issued as Section III in 1963. Section VIII Division 2 was issued in 1968 as the code applicable to the chemical industry.

[2] Design factor on fatigue strength

The purpose of fatigue evaluation in a design code for pressure vessels is thought to ensure that fatigue cracking does not occur in structures as an application of “defect-free design” concept. In fatigue evaluation, as described in Item [3] in Sect. 5.2.2, the allowable number of cycles, which is the denominator for the calculation of the usage factor, is obtained from the design fatigue strength or a design fatigue curve. Design fatigue curves are obtained from fatigue tests. The design factor is incorporated in the design fatigue curve.

Normally, a fatigue test is performed by controlling alternating strain at atmospheric pressure in the low-cycle region. The load applied to the specimen is controlled to keep the strain within a given range. The test uses unnotched specimens with a shape of hour glass type, etc. (some European codes like the CEN codes are based on test data obtained using welded joint specimens in addition to such plain specimens). As an elastically calculated stress, that is a stress calculated on the assumption of elastic body is used for integrity evaluation in design codes and standards, a design fatigue curve in the non-creep temperature range is given as the relationship between the stress amplitude and the allowable number of cycles (that is, an $S-N$ curve). The stress amplitude is obtained by multiplying the strain amplitude (a half of the strain range) in the fatigue test by the Young’s modulus of the material at room temperature.

How the design factor is incorporated in the design fatigue curve is described below. First, a best-fit fatigue curve is obtained as a regression curve for the fatigue fracture test data. Then, in comparison to the best-fit fatigue curve, a curve of the values obtained by dividing the number of cycles and the stress amplitude by the following design factors is defined as the design fatigue curve as shown in Fig. 5.8.

Design factor for the number of cycles: 20

Design factor for the stress amplitude: 2.

It is said that the design factor for the stress amplitude was introduced because with the design factor for the number of cycles only, the allowable stress amplitude does not significantly differ from the stress amplitude by the best-fit curve in the high-cycle fatigue regime.

These design factors were determined not only to obtain the “design minimum value” taking into account the test data scatter, but also to ensure that fatigue cracking does not occur at a particular allowable number of cycles, taking into account the difference between the fatigue test conditions and operating conditions of a structure, particularly the difference in the production method of a material (the structural codes require that the material of the specimen be the same, as much as possible, as that of the structure, including heat treatment), size, surface roughness, residual stress in the manufacturing process of the structure (the effect of mean stress on residual stress is incorporated in the design fatigue curves), environmental conditions between the specimen and the structure. However, they were empirically determined. Currently, a minor change is being considered in the discussion of environmental fatigue in reactor water environment.

It should be noted here that margins are incorporated not only in the design factor involved in obtaining design fatigue curves but also in the evaluation method specified in the structural codes. For example, the usage factor itself, including the mean stress effect, and the design number of operation cycles of events are conservatively determined.

Fatigue tests using full-scale structures were performed by the US Pressure Vessel Research Council (PVRC) and other organizations to quantitatively evaluate the conservatism of the design fatigue curves. See Ref. [6] for details. In the tests, fatigue cracking was not observed in most cases below the allowable number of cycles obtained from the design fatigue curve.

[3] Design factor on buckling

The principle of the rules for preventing buckling of vessels subjected to an external pressure or the method using external pressure charts against buckling based on factors *A* and *B* is described in Sect. 5.6.4. The values of the design factor *DF* in Eq. (5.47) in the same Section used in the method are provided below. Buckling is an instability phenomenon and therefore is strongly affected by the configuration of the structure and the type of load that causes buckling. Different design factors are used depending on these effects when empirically determining the allowable external pressure. The following are typical values used in the current codes [7]:

Cylindrical shells subjected to an external pressure: $DF = 3$

Spherical shells subjected to an external pressure: $DF = 4$

Cylindrical shells subjected to an axial compressive load: $DF = 4$.

External-pressure charts based on factors *A* and *B* must be used when applying the Design by Formula approach to structures subjected to an external pressure.

For the reactor containment vessel, it is a key point to assure integrity for seismic and other loads. These loads are often asymmetric and not uniform in the axial direction. Therefore, evaluation based on a generalized analytical method was essential, instead of the method for evaluating buckling based on theoretical solutions as given in Sect. 5.6.4. The ASME Boiler and Pressure Vessel Code Committee investigated rules for analytical evaluation of buckling, and issued N-284 in 1995 [8]. N-284 aims to provide general rules. Therefore, it presents a generalized method independent of the configuration of the structure and the type of the buckling load. The design factor varies with the service condition as for other failure modes and is specified as follows:

Service conditions A and B: $DF = 2.0$

Service condition C: $DF = 1.67$

Service condition D: $DF = 1.34$

The study was continued. A revised version reflecting the results of the study was issued as Code Case 2286 in 1998 [9].

5.4 Plant Operating and Service Conditions

Loading conditions as design specifications must be determined for the structural design of components. In the Design by Analysis approach that was proposed to allow high-stress design, as previously described, the response of a component to loads at all events that affect its structural integrity is analyzed in detail to show that the stress is below the allowable limit for burst, and other potential failure modes including fatigue failure. The design factor for the tensile strength is allowed to reduce to 3, under the condition that these requirements and others are met.

The loading conditions at the events for stress analysis vary from one plant to another. In some cases, if it is possible to determine the wall thickness to assure the structural integrity on the basis of structural response for a typical event with unique characteristics, in a transient response for example, structural integrity for other events can be demonstrated for the wall thickness determined. An example is a case that a set of overpressure protection devices (or, pressure relief devices) such as safety valves are installed, and it is guaranteed that overpressure arising at an accident does not exceed the specified set pressure. In this case, the design pressure may be determined on the basis of the pressure in the rated power operation, taking into account the above set pressure that is specified in accordance with design requirements on the overpressure protection devices, and the sizing of the vessel is made from the design pressure. Hence, the structural integrity is automatically ensured for all events including the accident. The detail is given in Sect. 5.4.1. In other cases, it may be necessary to evaluate all events for the specified occurrence frequency (even in these cases, a few typical events are sufficient for evaluation because the remaining events can be represented by them, based on a concept of enveloping by representative events).

If it is necessary to select several such events, it is desirable for reasonable structural design to classify them from a certain perspective. As briefly mentioned in Item [4] in Sect. 5.1, generally, the allowable conditions of a component in response to an event depend on its frequency of occurrence. For example, if it is necessary to consider an extremely unlikely event due to its severe hazard potential for the plant despite its low frequency of occurrence, it is not necessary to require that the response of the component to the event be an elastic response, which is required to restart without any examination. In an extreme case, it may not be necessary to consider the reuse of the plant if the functions (for example, not leading to large scale leaks) required during the event are performed and the event is safely terminated. The events to be selected are classified according to their occurrence frequencies, etc., focusing on the function to be performed during each event and the response of the component, and an appropriate allowable stress is determined for each category. A classification of functions based on what function should be performed as a plant system is called classification by plant operating condition. A classification of functions based on how the component should respond to the event to achieve the required function as a plant system is called classification by service condition.

It should be noted that it is reasonable for ensuring operability and reducing site personnel's exposure to avoid careful inspection and repair in the event of a likely event, or to allow restart without checks and examinations in such an event. In this case, components are stringently required to perform the required functions (for example, not inducing harmful damage to the component, and preventing leaks) as part of the system, and therefore, elastic design is required for them. However, elastic design is not necessarily required only for likely events.

First, the criteria for selecting events for structural design are described below. Then, two major approaches to event classification in the Design by Analysis approach are described.

5.4.1 The Basic Principle of Event Selection

As described in Item [6] in Sect. 5.1, two design approaches are available: the Design by Formula approach and Design by Analysis approach. If these approaches are difficult to apply, the Design by Experiment approach may be applied.

In the Design by Formula approach, wall thickness is calculated from design conditions. More specifically, the minimum required thickness is calculated by the specified formulae for the thickness so that the basic stress due to the design pressure is equal to or below the allowable stress at the design temperature. Recently, in some codes and standards, cyclic loadings such as thermal loading as fatigue damage evaluation need to be considered even for Design by Formula approach.

The Design by Analysis approach was proposed to solve the problems of manufacturing limitations resulting from increased thickness due to increased working pressure while ensuring safety, and for strictly assessing damages induced by cyclic loadings, etc., for ensuring an extremely high level of reliability. In order to justify

the reduction of the design factor (for the tensile strength) from 3.5 (4 when the proposal was made) to 3, it is required to perform integrity evaluation to prevent failures due to cyclic loadings, including thermal loading, and to evaluate all events, including accidents as well as normal operation.

Why all events need to be evaluated is discussed below.

Nuclear reactor components, particularly reactor coolant pressure boundary components, require a very high level of quality. Overpressure can be caused by events unique to nuclear reactors such as occurrence of boiling in reactor core that is caused by erroneous withdrawal of a control rod, as well as by trouble with pressure sources such as pressurizers. In basic design of facilities for design basis events, safety provisions should be provided to meet safety assessment acceptance criteria. For example, overpressure protection devices should be appropriately set up so that the pressure acting on the reactor coolant pressure boundary should not exceed 1.1 times the maximum allowable working pressure (or design pressure) at abnormal operational occurrence events, while at design basis accidents, it should not exceed 1.2 times the maximum allowable working pressure. A safety review of the basic design is conducted to meet the above requirements, and subsequently, at the stage of approval of design and construction plan, the detailed design such as structural designs of components is assessed if it follows the basic design. For these reasons, in the structural design, it is reasonable to classify each cause of overpressure as a plant operating condition and evaluate structural integrity for each condition.

On the other hand, for many pressure-retaining components of general plants, a design concept that the component design is carried out for design conditions only is employed, even if the Design by Analysis approach is applied (i.e., all events should be evaluated), owing to the following reasons. Such a design concept may be called the “design condition-based concept.” Causes of overpressure are clearly understood, and the overpressure can be controlled by means of the overpressure prevention devices of which the minimum discharge area, set pressure, etc. are determined from the level of overpressure clearly understood by applying well defined structural codes and standards such as JIS (Japanese Industrial Standards). For these cases, if the overpressure protection devices are designed from the overpressure causes clearly defined, we do not need to make any structural integrity assessment for overpressure at accidents. We need to make sizing of the component only for the design pressure. Of course, mechanical loading and other loading, such as earthquake loads and wind loads are required to assess, for which the structural integrity cannot be guaranteed by the pressure resistance design.

It should be noted here that even if an overpressure beyond the expected (or designed) happens, there may be still a sufficient pressure margin to cause burst. Of course, as mentioned earlier, a design factor as a margin has been determined to consider several uncertainties such as human errors. After such an unexpected overpressure occurred and the accident terminated safely, examinations/ testing/ inspections and follow-up activities like repair will be made for reuse.

The design condition-based design concept can be effectively applied to pressure-retaining components of general plants even if the Design by Analysis approach is adopted. For nuclear reactor components, however, the overpressure protection

devices are required to demonstrate to properly function for overpressures unique to nuclear reactors and at design basis accidents. In addition, as described in detail in Sect. 5.4.2, several assumptions to be made in safety design, such as a premise of pipe rupture in only one place at LOCA or prevention of secondary induced rupture of the piping system should be guaranteed. This is because design requirements of engineered safety features such as a reactor containment vessel, for example, the design pressure, steam condensation capacity of suppression chamber, etc. are based on these assumptions. Accordingly, the structural integrity of nuclear reactor components should be assessed for all events including anticipate operational occurrences and design basis accidents.

In doing so, it is important to determine an appropriate level of quality corresponding to the importance of functions required at each event, and hence to define adequate values of design factors. It is, therefore, reasonable to classify into several plant operating conditions in performing structural integrity assessment.

In summary, special and different requirements on the countermeasures against anticipated operational occurrences and accidents, respectively, leading to classification of quality level and therefore, classification of events and then plant operating conditions, are imposed as the safety design of nuclear reactors. As an example, special requirements for accidents as described above are provided in Article 27 of the “Ordinance on Technical Standards for Commercial Nuclear Power Reactors and their Auxiliary Facilities” in Japanese Regulations, as follows.

Components constituting the reactor coolant pressure boundary should be designed to withstand such loading acting on these reactor coolant pressure boundary components as an increase in loading that is induced by shocks due to collapse of a facility relating to the primary cooling system, and due to other abnormal events, and by a change in reactivity of the core, and others.

5.4.2 Selection of Events for Evaluation in the Design by Analysis Approach

[1] Two different approaches to event classification

Since the Design by Analysis approach requires the structural integrity assessment for all failure modes and all events, possibilities of fatigue failure and ratchetting due to cyclic load applications shall be explicitly evaluated. It should be noted here that concerning failure modes due to cyclic loading, rarely occurring events can be excluded from events for the integrity assessment, because fatigue damages developed at these events are not significant under the primary stress conditions to satisfy the primary stress intensity limits. In short, events of plant operating conditions III and IV, approximately corresponding to design basis accidents can be excluded from damage evaluation due to cyclic load applications, such as fatigue damage evaluation.

Similarly, if the test pressure value of the pressure tests exceeds the maximum allowable working pressure (or design pressure) and a number of their pressure tests is limited to 10 times or less, failure modes due to cyclic load applications,

typically fatigue failure, need not be evaluated, because the fatigue damage due to these pressure tests is negligibly small. These pressure tests are independently categorized as the “testing condition” of the plant operating conditions. Pressure tests other than those categorized as the “testing condition,” that is those whose test pressure does not exceed the maximum allowable working pressure (approximately identical to 1.1 times the operating pressure) and the number of tests is 10 times and more after commencement of operation. It should be noted here that incidentally, the “testing conditions” defined as service conditions include pressure tests of the latter case. Do not confuse the “testing condition” of the service condition classification with the “testing condition” of the plant operating condition classification.

On the other hand, for primary stress, which can cause failure (for example, burst due to overpressure) under a single load application, currently there are the following two approaches to selecting events for evaluation, as described above:

- (1) Investigate all events to be evaluated in safety design and others.
- (2) Select events representing design conditions from significant events and perform evaluation only for design loadings due to the events.

Table 5.5 shows the events to be evaluated in the Japanese and US codes specifying the Design by Analysis approach. The definitions of plant operating conditions and service conditions are described in Item [2] below.

Table 5.5 Selection of events for evaluation in the codes based on the Design by Analysis approach

Events to be evaluated	Evaluation of all events in limiting primary stress	Evaluation of selected events representing design loading in limiting primary stress
Codes and standards	United States: ASME Boiler and Pressure Vessel Code, Section III, Division 1, Subsection NB, Class 1 components Japan: JSME Codes for Nuclear Power Generation Facilities, Rules on Design and Construction for Nuclear Power Plant, Class 1 Components	United States: Typically, ASME Boiler and Pressure Vessel Code, Section VIII, Division 2 Japan: Typically, Japanese Industrial Codes JIS B 8266:2003 “Codes for Construction of Pressure Vessels”
Primary stress limits	Events* that cause loading under testing conditions or service conditions A to D	Events that define design loading** and pressure testing that defines loading under testing conditions
Secondary stress and peak stress (or total stress) limits	Events* that cause cyclic loading classified as operating loading (loading under service condition A and B)	Events that cause cyclic loading classified as operating loading (loading due to service condition A and B)

* Design loading is determined to envelop loads under service condition A. Therefore, the evaluation is not performed for loads under service conditions A and B.

** Design loading includes seismic and wind loads. However, the allowable stresses for these loads are increased by 20%.

Evaluation of all events is typically required by the codes applicable to Class 1 components (in particular, class 1 vessel and piping) of nuclear reactor facilities. The reason for this, which is briefly described above, is described below in detail.

[2] Selection of events for evaluation in the Design by Analysis approach for nuclear reactor components

The basic design of a nuclear reactor is reviewed in a safety review and its detailed design is reviewed in the process of design and construction plan approval.

In the basic design, representative events (so-called design basis events) are selected as the safety design basis, taking into account the concept of defense-in-depth and the probability and consequence of the event in safety design and assessment. As briefly described in Sect. 1.2, the concept of defense-in-depth is introduced here in detail.

The concept of defense-in-depth:

First, the “first stage safety measures” are taken in order to maintain normal operations and to prevent occurrence of abnormal operation and failures in conservative ways.

Second, if the “first stage safety measures failed or the normal operation fails to control, the subsequent stage safety measures (or the “second stage safety measures”) come into play for ensuring safety of the reactor facility. The “second stage safety measures” aim at preventing the evolution of abnormal operation and failure into accident, and bringing the reactor back to normal operation conditions as soon as possible, by means of controlling the abnormal operation and detecting the failure.

Third to fifth, assuming a failure of the “second stage safety measures”, the “third stage safety measures” shall be taken to control the accident within the design basis. Consecutively, the “fourth stage safety measures”, and then the “final or fifth stage safety measures” shall be provided.

In detail, the first three stages of safety measures are provided to protect people against abnormal radiation damages as well as the environment, and to return the reactor to a safe state. Since design basis events shall be selected for which the first three stages of safety measures must be designed to meet their functional goals, facilities to be installed as the first three stages of safety measures are termed “facilities for design basis events.” If the first three stages of safety measures fail, evolution to more serious conditions or a so-called severe accident, etc. may happen. The “severe accidents, etc.” include not only a severe accident at which a severe core damage is caused (however, a significant radioactive release, probably requiring off-site protective actions, will be prevented) but also an accident, other than anticipated operational occurrence and design basis accident, which has likelihood of causing a severe accident. In this connection, a defensive concept of many Japanese castles, which were constructed in sixteenth and seventeenth centuries, was based on a kind of defense-in-depth concept.

The design basis events are classified into three plant states comprising normal operation, anticipated operational occurrence and design basis accident. To cope

with the design basis accidents engineered safety features are provided, and then to prevent evolution to severe accidents, etc. Design conditions of these features are mainly established from responses at the design basis accidents, owing to MS (mitigation systems) components. For example, the maximum allowable working pressure and temperature of the reactor containment vessel are established from the response at LOCA. It should be noted here again that at LOCA as the design basis accident, rupture of the primary cooling system piping at more than one place is not assumed. Hence, loading generated by the rupture of piping at one place does not cause any significant damage of the other portions of the primary piping and do not cause further rupture of the piping.

State of a component to be achieved at each plant operating condition basically differs depending upon occurrence frequencies of events that are categorized in each plant operating condition. Limit values for planned normal operations (categorized as plant operating condition I) and for likely events (categorized as plant operating condition II) are more strictly established than those for accidents (plant operating conditions III and IV). An elastic design of component is required for plant operating conditions I and II. An elastic design means that primary stresses are limited within the elastic regime (or below the yield strength).

On the other hand, for such an extremely unlikely event like LOCA that are not considered to occur throughout the plant life, restart is not usually required without any examinations and repair after the event is safely terminated. A certain amount of plastic deformation is allowed to be developed if the safety functions required are achieved until terminating the event.

On the basis of the above considerations, the plant operating conditions are generally defined as follows:

(a) Operating condition I (normal conditions)

Condition under normal operation; more specifically, scheduled operating conditions or switching from one scheduled operating condition to another.

(b) Operating condition II (upset condition)

It should be noted here before discussing the definition of this term that in Japanese regulation on structural design, the term of “abnormal condition” was used for meaning the “upset condition.” However, in the field of safety design and evaluation, the “abnormal condition” was used to denote a group of design basis events that unify the anticipated operational occurrences and design basis accidents. In order to avoid confusion, terms of the plant operating conditions changed to “operating conditions I, II, III, IV and testing condition” in Japanese regulation.

Operating condition II means the condition likely to occur during the design life of nuclear reactor facilities, in which a single failure, a single operator error or others causes a deviation from the normal operating condition; conceptually equivalent to anticipated operational occurrence. It should be noted here again that its legal definition in Japanese regulation is different from the above, and that is not technical representation but political, in order to include other events than those in operating conditions I, III, IV and testing condition.

(c) Operating condition III (emergency condition)

Condition in which damage is developed by a failure of nuclear reactor facility or an abnormal operation, and an emergency shutdown of the reactor is required to repair the damaged portion; less likely events under the operating condition II (upset condition) defined in (b) above and conceptually equivalent to accidents and including some events of anticipated operational occurrence.

(d) Operating condition IV (faulted condition)

Condition in which abnormal conditions assumed in the safety design of nuclear reactor facilities are present; it is needless to say that significant damage occurs, requiring an emergency shutdown for repair. A failure of a prevention system (PS) within the range postulated in the safety design is assumed a priori to evaluate the validity of the design of a mitigation system (MS) as a defense-in-depth strategy for nuclear safety. The purpose of evaluating operating condition IV is to ensure that the failure remains within the range postulated in the safety design.

As mentioned above, the testing condition is defined in addition to the operating conditions I, II, III and IV. This is the pressure-testing events in which the test pressure exceeds the maximum allowable working pressure (or design pressure). Limiting the number of testing condition events within 10 times, an evaluation on failures by cyclic load applications such as fatigue damage evaluation need not be implemented. A test pressure of pressure testing after commencement of operation is 1.1 times the operating pressure (pressure at the rated power operation). These pressure-testing events are classified as operating condition II and evaluation on failures by cyclic load applications should be performed. In a case of PWR, the operating pressure is approximately 15.5 MPa, while the maximum allowable working pressure is about 17.5 MPa. Hence, 1.1 times the operating pressure can be said to be equal to or lower than the maximum allowable working pressure.

Contrary to the above-mentioned basic relationship between the state of a component to be achieved and the occurrence frequencies of events, or the plant operating conditions, a reactor containment vessel is required to achieve so stringent functions at LOCA as to confine radioactive materials (fission products) discharged into the vessel within an allowable rate of leakage to the environment (the leak tightness) as well as to withstand the internal pressure of the vessel (pressure resistance). The allowable leakage rate is approximately 0.1%/day. Such strict leakage is possible to attain the elastic design for LOCA loads. The leakage rate of 0.1%/day is the leakage rate from the vessel of 0.1% by its internal volume per day. Summarizing the above, although LOCA is classified as the plant operating condition IV, the reactor containment vessel shall be designed to withstand the LOCA loads as strictly as those in the operating condition I.

Wall thickness of a Prevention system (PS) component such as the reactor pressure vessel is practically determined from the design conditions on the basis of normal operating conditions, i.e., the plant operating condition I, because the sustained loads acting in the normal operating conditions are governing. On the contrary, the reactor

containment vessel, a typical component of the engineered safety features classified as the Mitigation system (MS), is designed to attain the above stringent functions for a very unlikely accident of LOCA, that is its wall thickness is determined so as to conduct the elastic design against loads in the plant operating condition IV. Such a difference in the relationship between the state of component and the plant operating conditions imposes the unique design of the reactor containment vessel different from designs of PS components such as the reactor coolant pressure boundary components in accordance with the plant operating conditions. This difference in design methodology may cause some confusion of deriving design conditions from the plant operating conditions and then resulted in a proposal of a new concept of service conditions.

The code of design and construction for nuclear power reactor equipment that is issued by the Japan Society of Mechanical Engineers (JSME) provides the following definitions of the service conditions. The ASME Boiler and Pressure Vessel Code Section III Subsection NCA provides slightly different rules from those of the JSME Code. These differences are shown in parentheses below, with referring to the Ref. [2].

(A) Service condition A

Conditions in which the components, etc. (i.e., components and their supports) are subjected to the maximum allowable working pressure and design mechanical loads in operating conditions to perform the significant functions (or in the performance of its specified service function [2]), which are designated in the Design Specification, etc.

(B) Service condition B

Conditions in which the components, etc. are subjected to pressure and mechanical loads that in the Design Specification, are designated as the “components, etc.” must ensure the integrity (or withstand the above loadings [2]) without damage (without repair [2]).

(C) Service condition C

Conditions in which the components, etc. are subjected to pressure and mechanical loads that in the Design Specifications, are designated as “permitting large deformations in areas of structural discontinuities and in others” (which may necessitate the removal of the component or support from service for inspection or repair of damage to the component or support [2]).

(D) Service condition D

Conditions in which the components, etc. are subjected to pressure and mechanical loads that in the Design Specification, are designated as “permitting gross general deformations” (with some consequent loss of dimensional stability and damage requiring repair, which may require removal of the component or support from service [2]).

Other than the above service conditions, testing condition (or test condition) is defined as mentioned above. The testing condition may correspond to the test condition that is defined as one of the plant operating conditions, but a little different because the testing condition as service condition classification is defined to include

pressure test events at which the test pressure is set as not more than the maximum allowable working pressure (in other countries than Japan, the design pressure).

To summarize the above, nuclear reactor components are required to ensure the integrity for events including design basis accidents as specified in technical standards of the “Reactor Regulation Act.” Correspondingly, in structural standards as specification codes and standards, the integrity assessment is required to implement for all events including design basis accidents. It should be noted here that it is not practical to determine the design conditions so as to envelop all events including the accidents. In the event of LOCA, which is practically very unlikely, there is the high possibility that loads exceeding those under normal service conditions may be applied at the accidents. The state of the components to be maintained during normal service conditions and an LOCA are significantly different. Therefore, for reasonable design, evaluation should be made for each service condition, instead of loading conditions that envelop all events to be postulated in design. This is why all events are evaluated and the events should be classified. Correspondingly, values of design factors shall vary in accordance with the state of component to be maintained in each service condition. This is the reason why for components of nuclear reactor equipment, to which the Design by Analysis approach is applied, all events are evaluated on the basis of event classification, as shown in Table 5.5. Table 5.6 summarizes the relationship between service conditions and the state of component (state of deformation and damage).

5.4.3 *Service Conditions and the Design Factor*

As described repeatedly, the allowable level of deformation and damage depends on the service condition. Therefore, it is reasonable to have different allowable range of stress, hence, allowable stress limits for different service conditions. This means that the design factor in Eq. (5.1) varies. In other words, the allowable stress limit range is wider for service conditions B, C, and D than for service condition A (or design conditions), or the allowable stress limits are increased for service conditions B, C and D. The basis for the increases in allowable stress limits is not clear. For Class 1 components using the Design by Analysis approach, as shown in Table 5.12 in Sect. 5.9.2, the relationship between the service conditions and the increases in allowable stress limits is very complex. To help understand the concept, the case of the Design by Analysis approach applied as an alternative approach to Class 2 components of nuclear reactor facilities in the ASME Boiler and Pressure Vessel Code is shown in Table 5.7. As you can see, the increase rate k over service condition A is 1.1, 1.2, and 2.0 for B, C, and D, respectively. What will the values in the table be like for Class 1 components?

Table 5.6 Concepts of service conditions and deformations allowed in their conditions for primary stress (intensity) limits (for design basis events) (This table is elaborated referring to the JSME Rules on Design and Construction for Nuclear Power Plants [1] and the relevant ASME Code [2])

Service condition	Objectives of stress limits for the service condition	Deformation allowed in the service condition or design concept for primary stress limits	Correspondence to plant operating conditions (a case of reactor pressure vessel as PS components)
A	The stress limits are specified as requirements for loading that may be applied during a normal service (or in the performance service functions specified to components, etc.).	Components, etc. should be maintained within the elastic regime (the elastic design).	Normal operating condition
B	The stresses may be allowed to exceed the limits for the normal service conditions or the design conditions, but the loading does not cause any damage requiring repair.	The elastic design	Upset condition
C	The stress limits allow such large deformations in areas of structural discontinuities that may necessitate the removal of relevant component or support from service for inspection or repair of damage to it.	To keep the elastic response in general regions	Emergency condition
D	The stress limits allow such gross deformations even in general regions, resulting in some loss of dimensional stability and damage requiring repair, that may require removal of the relevant component or support from service.	To allow plastic deformations in general regions	Faulted conditions

5.5 Component Classification

As described in Sect. 1.3 in Chap. 1, the purpose of the structural codes and standards is to achieve (harmonize) both the safety and economy of mechanical equipment. Since the level of quality is the direct concern of the structural codes, the level of quality required to achieve both safety and economy should be properly determined and rules should be established to achieve it. If the required level of quality significantly varies with the component, it is desirable to establish rules most suitable to each component. However, it is not reasonable to establish rules for each component. There are not as many different technologies as can be used to achieve a different level of quality in different stages (design, fabrication, examination, etc.). Therefore, it is practical to divide the components into groups by the level of quality and select the technology most suitable for each group.

Particularly for nuclear reactor facilities, the required level of safety importance largely depends on the component, and therefore, the difference in quality level is

Table 5.7 Concepts of amplification multipliers on the allowable stress criteria for service conditions A to D and hydrostatic/pneumatic pressure testing [10] (The Design by Analysis approach applied to Class 2 components in the ASME Boiler and Pressure Vessel Code)

Loading condition	Stress classification			
	General primary membrane stress	Local primary membrane stress	Primary membrane + bending stress*	Primary + secondary stress
Service condition A	$kS_m, k = 1.0$	$1.5kS_m, k = 1.0$	$1.5kS_m, k = 1.0$	$3.0S_m$
Service condition B	$kS_m, k = 1.1$	$1.5kS_m, k = 1.1$	$1.5kS_m, k = 1.1$	$3.0S_m$
Service condition C	$kS_m, k = 1.2$	$1.5kS_m, k = 1.2$	$1.5kS_m, k = 1.2$	Not applicable
Service condition D	$kS_m, k = 2.0$	$1.5kS_m, k = 2.0$	$1.5kS_m, k = 2.0$	Not applicable
Hydrostatic pressure testing	$0.9S_y$	Not evaluated	$1.35S_y^{**}$	Not applicable
Pneumatic pressure testing	$0.8S_y$	Not evaluated	$0.8\alpha S_y$	Not applicable

* For service conditions A to D, 1.5 is the shape factor for rectangular cross-section. For other than rectangular sections, can be used as the shape factor, instead of 1.5.

** $1.35S_y$ for membrane stress intensity $< 0.67S_y$; $2.15S_y - 1.2S_m$ for membrane stress intensity $\geq 0.67S_y$

[Notes]

1) In the table, S_y is the yield strength (yield point). For ferrite materials such as carbon steels and low-alloy steels, S_m is obtained as the least of the following :

- 1/3 of the minimum tensile strength at room temperature
- 1/3 of the design tensile strength at each temperature
- 2/3 of the minimum yield point (yield strength) at room temperature
- 2/3 of the design yield point (yield strength) at each temperature

2) Evaluation of fatigue damage is not included in the table.

quite large between components. The required level of quality is classified into an appropriate number of levels, taking into account economy and other factors, with emphasis on safety.

Technology desirable for each stage is selected for each level of quality, taking into account the characteristics of individual components. A combination of technologies through all stages can be optimized to reasonably achieve the required level of quality.

Component classification given in structural codes and standards for nuclear reactor equipment is implemented according to the “order of importance,” in considerations of requirements as system safety criteria as the highest priority, because of the severity of potential reactor hazard. The order of importance is defined as follows. Note that the following classification is applied to components of facilities for design basis events. The term “facilities for design basis events” is defined as facilities that are functioned at design basis events.

- (1) Components requiring the highest level of quality (or extremely high level of quality): reactor coolant pressure boundary components. They form the most important physical barrier for containing radioactive materials in the reactor core as well as the reactor coolant.

- (2) Components requiring the lower level of quality: the reactor containment vessel providing the last defense as an MS. They confine the radioactive materials in order to prevent undue exposure to residents even when a design basis accident like LOCA happens.
- (3) Components requiring the further lower level of quality: direct systems surrounding the reactor coolant pressure boundary (primary systems), or connecting to the reactor coolant pressure boundary
- (4) Components requiring the lowest level of quality, equivalent to that of fossil-fuel-fired power plants: indirect systems surrounding the reactor coolant pressure boundary (the secondary systems and their downstream systems).

Table 5.8 shows component classification for facilities for design basis events, as specified in the “Reactor Regulation Act.”

In the Japanese regulation, facilities that are functioned at severe accidents, etc. are legally prescribed in the response to lessons learned from the Fukushima Daiichi nuclear accident. They are called the “facilities for coping with severe accidents, etc.” in this text book. Figure 5.9 shows the classification of both facilities for design basis events and those for severe accidents, etc. In this figure, subdivided definitions of classification for facilities for severe accidents, etc., depending on portability (permanent or portable) is given. In this figure, we can see a unique classification of facilities for coping with severe accidents, etc. into two categories; One is those that are installed only for coping with severe accidents, etc., and another is that facilities for design basis events are utilized for coping with the severe accidents, etc., namely, to extend their usage to the severe accidents, etc. Further, the latter is subdivided into two subcategories; one is to utilize them without any change for coping with severe accidents, etc., and another is to change their design specifications for coping with severe accidents, etc. For instance, the design pressure and design temperature are increased to be functioned in more severe conditions than those at design basis events. The former is called in this textbook the “facilities for extended design basis events.” It is needless to say that minor changes may be made, even for those facilities for extended design basis events, because service environments differ between the design basis events and severe accidents, etc. The detail of facilities for coping with severe accidents, etc. are described in Sect. 5.10.

The ASME Boiler and Pressure Vessel Code Section III Division 1 is briefly described below. In the Japanese rules on design and construction [1], not only the design codes but also the other codes for all components of nuclear reactor facilities are compiled in one volume (recently, rules on welding, materials and maintenance (fitness for service) have been separated into individual volumes). On the other hand, the ASME Code is divided into sections for each component class. Further, rules on materials, welding, examination and testing, and maintenance (in-service inspection) have been separated into individual volumes. Table 5.9 shows the correspondence between the component classes in the ASME Code and in the Japanese codes, as well as the correspondence between the component classes and the sections in the ASME Code. In Japan, vessels, piping, valves, and pumps are classified as components. The term of “components, etc.” is also defined to involve supports for the components.

Table 5.8 Component classification of facilities for design basis events (This table is based on Article 2 of the “Ordinance on Technical Standards for Commercial Nuclear Power Reactors and their Auxiliary Facilities,” and is referred to the MITI Notification 501 “Technical Standards on Structures, etc., of Nuclear Power Reactor Equipment” and ASME Boiler and Pressure Vessel Code Section III Division 1 [2].)

Component classification		Content (functions, etc.)
Primary item	Secondary item	
Class 1	Vessel	Components constituting the reactor coolant pressure boundary
	Piping	
	Pump	
	Valve	
Class MC*	Vessel	Reactor containment vessel and vessels connecting to this vessel that aim at preventing leakage of hazardous substances such as radioactive materials discharged from machinery or apparatus in the reactor containment vessel and vessels connecting to this vessel
Class 2	Vessel	•Components to indirectly have the possibility of causing radiation damage to public due to their collapse, malfunction or other abnormality, of equipment necessary to safely stop the nuclear power reactor or to ensure safety of nuclear power reactor facilities, in an environmental condition that is expected at and up to an event of design basis accident (ducts belonging to radiation control facilities or reactor containment facility (limiting to emergency gas treatment equipment) in the range from the penetration to the other isolation valve). •Components in the ranges from a class 1 component of a steam system located on the downstream side of the class 1 component to the stop valve closest to it, among components belonging to equipment relating to a circuit circulating the fluid for the main purpose of driving a steam turbine.
	Piping	
	Pump	
	Valve	
Class 3	Vessel	Vessels and piping (limiting to piping with the concentration of radioactive materials in the contained fluid of 37 mBq/cm ³ or more (when the fluid is a liquid, 37 kBq/cm ³) or piping with the maximum allowable working pressure (or design pressure) exceeding 0 MPa) other than class 1 components, class 2 components, reactor containment vessel and ducts belonging to radiation control facilities and reactor containment facility (limiting to emergency gas treatment equipment).
	Piping	
Class 4	Piping	Ducts belonging to radiation control facilities or reactor containment facility (limiting to emergency gas treatment equipment), with concentration of radioactive materials in the contained fluid of 37 mBq/cm ³ or more (excluding parts belonging to class 2 piping).

(continued)

Table 5.8 (continued)

Component classification		Content (functions, etc.)
Primary item	Secondary item	
Support structure (or support)	Core support structure (class CS)**	Structures to support the reactor core.
	Class 1 support structure	Structures to support class 1 components, reactor containment vessel (the abovementioned class MC vessel) or class 2 components, respectively.
	Reactor containment vessel support structure	
	Class 2 support structure	

* There are a metallic containment vessel and a concrete containment vessel as a reactor containment vessel. The metallic containment vessel is categorized as class MC vessel. The definition of the concrete containment vessel is basically similar to that of the metallic containment vessel.

** The classification CS denoted in the parentheses is according to the ASME Boiler and Pressure Vessel Code Section III Division 1²⁾.

Table 5.9 Correspondence between the component classes and the sections of these components in the ASME Boiler and Pressure Vessel Code [2], and also the correspondence to the JSME codes [1]

Component class in ASME Boiler and Pressure Vessel Code	Typical component	Section of ASME Boiler and Pressure Vessel Code	Component class in Rules on Design and Construction of Nuclear Power Plants, JSME Codes for Nuclear Power Generation Facilities
Class 1 components	Reactor coolant pressure boundary components	Subsection NB	Class 1 components
Class 2 components	Direct system components	Subsection NC	Class 2 components
Class 3 components	Indirect system components	Subsection ND	Class 3 components
Class MC	Metal reactor containment	Subsection NE	Class MC vessels
Class CS	Core support structures	Subsection NG	Core support structures
Supports for piping and other components	Supports	Subsection NF	Supports
Reactor internals other than Class CS	Reactor internals	May comply to Subsection NG	Not applicable
Not applicable	Part of radiation control system duct	Not applicable	Class 4 piping systems

5.6 The Rules for the Design by Analysis Approach (General Concept)

In the Design by Analysis approach, a stress analysis of a structure under evaluation in loading conditions for all of the potential failure modes and all events (actually, selected representative events) is performed for sizing and evaluation of structural integrity. Vessels have a variety of functions and are operated in various conditions. Examples of the functions are to contain and transport internal fluid, and to exchange heat between primary and secondary fluids. These functions differ from each other. Due to the complex interaction with surrounding components, they are subjected to a variety of loads. The number of vessels per system is relatively small. Therefore, configurations and dimensions of the vessel as well as their materials are different one by one, resulting in so-called “individual design manufacturing.” On other hand, piping varies in type and size, but basically their manufacturing can be categorized as a type of small kinds in large quantities. Further, kinds of loads are relatively limited, and are characterized by a small radial temperature gradient, dominant stress due to internal pressure and bending stress due to bending moment, etc. Accordingly, standardization of piping design has many advantages, differing from the vessel design. As a result, the Design by Analysis approach applicable to any generalized conditions is applied to vessels. The Design by Analysis approach applied to piping systems is simplified, taking into account their specific features. The generalized concept of the Design by Analysis approach suitable for vessels is described in this section. Application of the specific Design by Analysis approach to piping systems is described in Sect. 5.7. The practice of the generalized rules is described in Table 5.12 in Sect. 5.9.2. In the current design codes, rules are established for each type of stress instead of each failure mode. This is because as previously described, there is a nearly one-to-one correspondence between the failure mode and the type of stress, and the stress is the parameter used for evaluation. The concept of the Design by Analysis approach for each type of stress is described below.

It should be repeatedly noted here that stresses to be evaluated in design codes are elastically calculated stresses on the assumption of an elastic material, while in many cases, maximum loads or limiting loads as applied loads are calculated using a limit analysis on the assumption of an elastic-perfectly plastic material. Even in these cases, stresses that are developed in a structural element subjected to the limiting load, that is “resistance” R in Eq. (5.1) for deriving the allowable stresses, are calculated elastically.

5.6.1 *Primary Stress Limits*

[1] Basic philosophy

The following failure modes are prevented by limiting primary stress.

- (1) Ductile fracture and burst
- (2) Plastic collapse
- (3) Gross plastic deformation (distortion)
- (4) Excessive plastic deformation.

Failure modes (1) to (3) are prevented by limiting general primary membrane plus bending stress. The allowable stress limit to prevent failure of these modes is determined by dividing the design tensile strength and the design yield point by proper design factors. In limiting primary stress in the Design by Analysis approach, to be exact, the stress intensity is limited instead of the stress itself. As mentioned in Item [1], in Sect. 5.2.2, this is because in a three-dimensional stress field, yielding and fracture are not always caused by a stress component in a certain direction (for example, the hoop stress under internal pressure) but have a so-called three-dimensional effect. Then, many studies have been done on what kind of expression the yielding and fracture can be represented as the equivalent stress. As mentioned in Item [1], in Sect. 5.2.2, the stress intensity criterion (or the maximum shear stress criterion proposed by Henri E. Tresca) has been adopted as the equivalent stress in structural codes and standards. The reason why this criterion has been adopted is that from the late 50s and to the 60s when the Design by Analysis approach was established, several formulae had been proposed for stress calculation by hand or calculators on the basis of knowledge of strength of materials. The stress intensity criterion can predict the yielding condition on the safe side of experimental data. Unless otherwise stated, the term “stress” is used below to mean “stress intensity.” Failure mode (4) is prevented by limiting local primary membrane plus bending stress. The local primary membrane plus bending stress is limited by the allowable stress determined by dividing the design yield point by a proper design factor.

From the historical background as mentioned earlier, the basic concept of design factor is determined to prevent ductile fracture (or actually boiler explosions) and to ensure elastic behavior during normal operation (service condition A). Ensuring elastic behavior is to prevent through-thickness yielding. If a primary stress exceeding the yield stress is allowed at normal operation, as is mentioned in Item [2] below, permanent deformation due to dislocation movements is allowed and other issues will be required to investigate.

In the European codes, the allowable stress to limit primary stress is determined only from the yield stress. However, it is not appropriate if there is no limitation on the materials used. Since ductility and strain hardening vary with the material, it is inappropriate to determine the allowable stress for materials with low ductility or high yield-to-tensile ratios only from the yield stress. Therefore, in the structural codes in Japan and the United States, typically the ASME Boiler and Pressure Vessel Code and the JSME Rules on Design and Construction for Nuclear Power Facilities,

the allowable stress is determined from both the design yield point and the design tensile strength (and also the minimum yield point and minimum tensile strength).

[2] General primary membrane plus bending stress limits

(1) Allowable range (in practice)

As previously discussed, let S_m be the allowable stress under normal operating conditions (service condition A or the design condition) when there is no bending component, or simple axial loading conditions are applied. This prevents burst and plastic tensile instability.

If a bending stress component or a bending moment is added, then the focus is to prevent plastic collapse. In the ASME Boiler and Pressure Vessel Code and the Japanese codes, the term “plastic collapse” refers to significant plastic deformation of the structure as a whole. The term “plastic tensile instability” refers to the point at which the slope of the load–displacement curve, which is typically obtained from a tensile testing, is reduced to zero, and at this point, unbounded deformation is developed without an increase in load. The reason why as the failure mode of a structural element subjected to a bending moment the plastic collapse is newly defined, but different from the ductile fracture following the plastic tensile instability, which is caused by an application of membrane stress, is the structural behavior under bending moment different from that under the membrane stress. The structural behavior under bending moment is as follows; first, yielding occurs on the tensile and compressive surfaces, and then a yielding area where a plastic deformation is developed is extending from the surfaces to the inside. As increasing the bending moment, a crack initiates on the tensile stress surface, and propagate to the inside, leading to the ductile fracture without causing the plastic tensile instability.

It is hard to analyze such structural behavior starting from a large plastic deformation to cause a crack initiation up to ductile fracture. If the material is elastic-perfectly plastic, the yielding of the entire cross section (tensile yielding on the tension side and compressive yielding on the compression side) is present, and a plastic hinge is formed, as the asymptotic situation. This is considered to be a case of plastic collapse. In such a case of plastic collapse developed in elastic-perfectly plastic material, deformation increases unboundedly if the plastic hinge is formed, and therefore, plastic collapse is also a kind of plastic instability.

Summarizing the above discussion, it is not easy to predict ductile fracture condition of a structural element made of work-hardening material, and subjected to bending moment. If an elastic-perfectly plastic material is assumed, the bending moment to form the plastic hinge is easily predictable by means of formulae given by the strength of materials. This bending moment is the maximum at which deformations of a structural element increase unboundedly, and causes ductile fracture. Therefore, the formation of plastic hinge can be defined as a failure for an ideally elastic-perfectly plastic material. To base on the above idea, as the starting point, consideration of deformations of piping and other components leads to generally define gross deformations following development of large deformations in local

areas as a failure. Accordingly, the plastic collapse may be defined as occurrence of significant deformations in a structural element as a whole, or so-called gross deformations.

For developing expressions as a rule in structural codes and standards, first, the plastic collapse load is calculated based on the conservative assumption that the material is elastic-perfectly plastic. The stress under this plastic collapse load (precisely speaking, limit collapse load) is called the limit stress. The combination of membrane and bending stresses used as the limit stress is obtained. The allowable stress range is obtained by dividing the limit stress by a design factor. It is important to note that the above critical bending moment or the critical combination of membrane and bending stress or axial tension and bending moment will be less than the bending moment or their combination at which the crack initiates on the tensile surface in a real material with work hardening. The critical bending moment or their combination forming the plastic hinge may be conservatively defined as the moment or their combination at a newly defined mode of failure,

The analytical model is shown in Fig. 5.10a, b. We assume an elastic-perfectly plastic rectangular beam, with the yield point S_y given in the structural codes, subjected to an axial tensile load F and a bending moment M as shown in Fig. 5.10a. The distribution of the limit stress under plastic collapse loading conditions is as shown in Fig. 5.10b. The plastic collapse load is the limit beyond which plastic collapse occurs, calculations of which are given in detail in item (3). Under these loading conditions, the axial tensile load F_c , the bending moment M_c , and the stress meet the following equilibrium equations:

$$\begin{aligned} F_c &= H \int_{-t/2}^{+t/2} \sigma(y) dy = H \cdot (-2xS_y) \\ M_c &= H \int_{-t/2}^{+t/2} \sigma(y)y dy = H \cdot S_y \cdot \left(\frac{t^2}{4} - x \right) \end{aligned} \quad (5.4)$$

where x is the point where the stress changes from a positive to a negative yield stress.

In the codes, the stress to be evaluated is obtained by elastic calculation, that is, the stress is calculated assuming an elastic material. Now, we consider a structural element subjected to a combined axial tensile force F_c and bending moment M_c that is causing the plastic collapse. These loads are derived on the assumption of elastic-perfectly plastic material. Stresses of this element subjected to these force and moment can be calculated on the assumption of elastic material. These stresses correspond to the resistance R for the failure mode of plastic collapse in Eq. (5.1). The membrane stress σ_m and the bending stress σ_b from elastic calculation are given by the following equations obtained from Eq. (5.4).

$$\begin{aligned} \sigma_m &= \frac{F_c}{H \cdot t} = -2 \left(\frac{x}{t} \right) S_y \\ \sigma_b &= \frac{M_c}{Z} = \frac{3}{2} S_y \left[1 - \left(\frac{2x}{t} \right)^2 \right] \end{aligned} \quad (5.5)$$

where σ_b is the elastically calculated maximum stress on the surface on the tensile side.

Then, in the limit stress state, σ_b/S_y is given by the following equation in terms of σ_m .

$$\frac{\sigma_b}{S_y} = \frac{3}{2} S_y \left[1 - \left(\frac{\sigma_m}{S_y} \right)^2 \right] \quad (5.6)$$

$\sigma_m + \sigma_b$ is given by the following from the above equation.

$$\frac{\sigma_m + \sigma_b}{S_y} = -\frac{3}{2} \left(\frac{\sigma_m}{S_y} \right)^2 + \frac{\sigma_m}{S_y} + \frac{3}{2} \quad (5.7)$$

Figure 5.11 shows this limit stress state. In the figure, the load-carrying capacity for $\sigma_m + \sigma_b$ seems to increase for $\sigma_m/S_y \leq 0.4$ with increasing primary membrane stress, but it does not. In the limit stress state, $(\sigma_m + \sigma_b)/S_y = \sigma_b/S_y = 3/2$ for $\sigma_m = 0$. If a limit stress increases proportionally as σ_m increases under the constant bending moment corresponding to $\sigma_b/S_y = 3/2$, simply, $(\sigma_m + \sigma_b)/S_y = \sigma_m/S_y + 3/2$. This means a straight line. However, actually, with increasing σ_m , the limit stress decreases faster than this straight line. This is as seen from Eq. (5.6) because the plastic hinge, at which yielding over the entire cross section occurs, is formed in the application of smaller bending moment than the constant bending moment corresponding to $\sigma_b/S_y = 3/2$.

(2) The allowable stress range in the codes

In the ASME Boiler and Pressure Vessel Code and other codes, the rules for both membrane stress alone and for membrane plus bending stress are established based on the limit stress obtained from the so-called limit design theory that assumes an elastic-perfectly plastic material. As repeatedly mentioned, the limiting stress or the limit stress above is derived as the elastically calculated stress, while the limiting load is obtained on the assumption of elastic-perfectly plastic material. Further, the membrane stress cannot be properly limited only by limiting membrane plus bending. Consequently, both stress limits are required to satisfy. The allowable range of membrane stress component as primary stress, to be exact, the general primary membrane stress intensity P_m under design loading conditions or service condition A is below S_m as described above. More specifically, the regarding the yield point (or yield strength, yield stress), a determinant of S_m , the allowable range is limited to below $(2/3)S_y (= S_y/1.5)$, that is with the margin of 1.5 below the yield strength. We can see several reasons why the margin of 1.5, i.e., the design factor of 1.5 is adopted. A reason may be that at that time when the margin of 1.5 was decided, the pressure of pressure testing was 1.5 times the design pressure. No gross yielding was allowed even at the pressure testing. The tensile strength has already been described above and is not described here again. Membrane plus bending stress, to be exact, the general primary membrane plus bending stress intensity is limited to keep the maximum stress below S_y under design loading conditions.

The reason for limiting the stress intensity to prevent a primary stress from exceeding the yield strength is that the following situations, for example, arises if the yield strength is exceeded.

- Permanent plastic deformation occurs.

Actual material (work hardening is expected) enters the plastic region if a primary stress exceeds the yield strength. Movements and multiplication of dislocations and other dislocation mechanisms are caused by the plastic deformation. Vacancies and other defects initiate and grow to form voids as an increase in plastic deformation.

- At a pressure testing, a plastic deformation is developed.
- If applying a primary stress exceeding the yield strength to a structural element and then unloading the primary stress, a permanent plastic deformation remains in the element. As a result, yielding at the following steps will cause complicated responses.

Now, for simplicity, assume an elastic-perfectly plastic material. The stress cannot increase beyond the yield strength, and after reaching the yield strength, the deformation increase is assumed to stop at a certain point, for example, at the point A shown in Fig. 5.12, (In a load-controlled tensile testing, an increase in stress beyond the yield strength will cause necking and then unstable deformation, instantaneously leading to ductile fracture. Here, we assume to control a “primary load” to cause a primary stress in such a way that the primary load is maintained so as to stop at the predetermined deformation.) Next, if unloading begins as the primary stress, it decreases with decreasing the load. At the end of unloading process, i.e., if completely unloaded, the load becomes zero, and correspondingly the primary stress decreases to zero. However, the permanent plastic deformation remains due to plasticity. The residual plastic deformation exists. On the other hand, consider the secondary stress response, for comparing to that of the primary stress.

When the structural element is cooled to contract, it must be stretched to remain as the original geometry, for example. When the material enters the plastic region due to an excessive secondary stress, tensile plastic deformation is developed. When unloading begins, or when cooling is stopped and a heating up begins, as shown in Fig. 5.12, the secondary stress follows the same path as that of the primary stress. However, when the material returns to the initial temperature, the tensile plastic deformation remains, and therefore, it does not return to the original geometry. As a result, compression must be applied to cancel out the tensile plastic deformation. When returning to the original geometry of the material, a compressive residual stress remains in the material.

Thus, when the primary or secondary stress is applied alone, the structural response in subsequent steps is different from each other. When a combined primary and secondary stresses are applied, and the primary stress exceeding the yield strength, a complicated response is easily imagined. As expected in the above discussions, the primary stress is limited so as to remain in the elastic regime in the design conditions or service condition A with an adequate margin. Of course, this limitation

is set for not only the above reasons but also other reasons of easy restart, etc., as mentioned in Table 5.6.

Next, what about a design factor for a limit on combined membrane and bending stress (called membrane plus bending stress), precisely speaking, on primary membrane plus bending stress intensity? The basis of design factor in the design conditions and service conditions A is to limit the maximum primary stress below S_y even for membrane plus bending stress. Under membrane stress only, if on the assumption of elastic-perfectly plastic material, the applied stress reaches or exceeds the yield strength, the plastic tensile instability occurs, instantaneously leading to ductile fracture. Correspondingly, the stress intensity on the assumption of elastic material, or the primary membrane stress intensity P_m reaches or exceeds the yield strength, the plastic tensile instability and then ductile fracture occurs. In other words, the plastic tensile instability and then the ductile fracture occurs for $P_m = 1.0S_y$. However, under combined membrane and bending stress, the plastic hinge criterion to cause failure is not $P_m + P_b = 1.0S_y$ due to the nature of bending stress. Figure 5.11 gives the answer for a rectangular cross section. This can be generalized to say that due to the nature of the shape factor described below, the bending moment-carrying capacity is higher than $P_m + P_b = 1.0S_y$. This effect depends on the shape factor of cross section, α , as is described later.

Now, we consider the design factor. The failure condition is expressed by the limit state curve given in Fig. 5.11. It seems reasonable to use the same design factor for all combinations of membrane and bending stresses. If so, the allowable stress range is determined from Eq. (5.7) by introducing the constant design factor. As you can see in the figure, the equation is complex to use for calculation. Therefore, in the current design codes, the rectangular shaded area in Fig. 5.11 is used as the allowable stress range. That is, the failure is assumed to occur conservatively when reaching the following expression:

$$P_m + P_b = \alpha S_y \quad (5.8)$$

Accordingly, the allowable stress range is expressed as follows, by replacing S_y in Eq. (5.8) with the design stress intensity S_m for the design condition loading,

$$P_m + P_b \leq \alpha S_m \quad (5.9)$$

If S_m is identical to $(2/3)S_y$, the inequality (5.9) becomes $P_m + P_b \leq \alpha \frac{2}{3} S_y \leq S_y$, because α is assumed to be equal to or less than 1.5. For a rectangular cross section, $\alpha = 1.5$. This means that the primary membrane plus bending stress intensity under the design conditions is limited equal to or less than S_y .

Summarizing the above discussion, inequality (5.9), also as given in Table 5.10 expresses the design allowable stress range for a structural element with a plain configuration, remote away from structural discontinuities, subjected to a combination of tensile axial load and bending moment under the design conditions or service conditions A.

Table 5.10 Summary of general primary membrane plus bending stress intensity limits

Classification of primary stress in general part	Failure in a real material (work-hardening material)	Failure on the assumption of elastic-perfectly plastic material with yield strength S_y	Limits on elastically calculated stresses for application of the load to induce the stress given in the left-side column on the basis of limit design	Stress limits for design loading provided in codes and standards ^{1), 2)}
Membrane stress	Occurrence of plastic tensile instability at tensile strength S_u , leading to necking and finally ductile fracture	Occurrence of plastic tensile instability at S_y , leading to necking and finally ductile fracture	General primary membrane stress intensity limit: $P_m \leq S_y$	$P_m \leq S_m$
Bending stress	•The crack initiation occurs at a maximum stress, namely, on the surface on the tensile stress side, leading to ductile fracture. Until the crack initiation, the bending is stably proceeding. •The possibility of local buckling for example, on the compressive side of a thin-walled cylindrical tube •Brazier effect	Application of a critical bending moment to form a plastic hinge (collapse load) M_c causes unbounded bending deformation.	Primary bending stress intensity: $P_b \leq 1.5S_y$, or $P_b \leq \alpha S_y$	$P_b \leq 1.5S_m$ or $P_b \leq S_m$
Membrane plus bending stress	•If a membrane stress dominates or controls the stress distribution, the failure due to the membrane stress mentioned above occurs. •On the other hand, if a bending stress dominates or controls the stress distribution, the failure due to the bending stress occurs first, and may be followed by the failure due to the membrane stress because of reduction in cross section in some cases.	For a combined loading of axial tensile load to cause plastic tensile instability and bending moment to form a plastic hinge, the similar failure situations to the left-hand column are expected.	Limit stress diagram for a combination of general primary membrane stress and primary bending stress	Both the stress limits below shall be satisfied: •$P_m \leq S_m$ •$P_m + P_b \leq 1.5S_m$ - or •$P_m + P_b \leq \alpha S_m$

(3) Shape factor effect

Here, we consider a relationship between a bending moment at incipient yielding, M_y , that is the bending moment to cause initiation of yielding at the maximum stress point or on the surface, and a bending moment to form a plastic hinge, M_c , as the limiting moment. We introduce the shape factor α as defined $M_c = \alpha M_y$. In the following, we assume a beam with rectangular cross section, subjected to a pure bending moment on the assumption of the elastic-perfectly plastic material. Other geometries of cross section should be considered when reading the following description.

The equilibrium condition for pure bending is expressed as follows:

$$\begin{aligned} F &= H \int_{-t/2}^{+t/2} \sigma(y) dy = 0 \\ M &= H \int_{-t/2}^{+t/2} \sigma(y) y dy \end{aligned} \quad (5.10)$$

where F is the axial tensile load, M the bending moment, and H and t the width and thickness of the beam, respectively.

For elastic-perfectly plastic materials, the relationship between the bending moment and the angular displacement is as shown in Fig. 5.13. Figure 5.13d shows the axial stress distribution across the beam wall under the limit stress state condition as shown in Fig. 5.11. The limit stress state means the formation of the plastic hinge, that is the fully plastic section or full cross section yielding. As well known, the strain distributions are linear because of a plane cross section assumption. In Fig. 5.13, the axial stress distributions are depicted by the solid lines, while the bending strain distributions are by the two-dot chain lines. As the bending moment increases, the stress distribution changes from (a) through (c), approaching the asymptotic state (d). Figure (b) shows the stress and strain distributions for the incipient yielding bending moment M_y .

As the moment increases, the yielding region is extending from the surface into the “core” (the central portion), while the core region remains elastic. Because of the assumption of the elastic-perfectly plastic material, the stress in the yielded region is constant at S_y .

As shown in Figure (c), it is possible increase the bending moment beyond M_y , because of the elastic stress region in the core. In the state (d), the fully plastic section as the asymptotic and ultimate state, unbounded angular displacement is developed, and therefore, it is impossible to further increase the bending moment. This situation is called the plastic collapse, as mentioned above. The moment is the maximum, as denoted M_c .

The shape factor α is defined as follows in terms of the ratio of the plastic collapse (or plastic hinge) moment M_c to the initial yielding moment M_y producing initial yielding in the extreme fibers of the section:

$$\alpha = \frac{M_c}{M_y} \quad (5.11)$$

Here, consider the implication of the above expressions from a viewpoint of structural codes. The membrane stress intensity P_m is limited below the yield strength, on the assumption of the elastic-perfectly plastic material. Reaching S_y means to cause the ductile fracture. It can be said that for membrane stress, the first yield load is identical to the ductile load. Incorporating the design factor results in below S_m . On the other hand, M_y is defined as the bending moment at which the maximum value of elastically calculated stress distribution or the bending stress intensity P_b reaches S_y . M_y can be said the first yield point moment. The moment to cause the plastic collapse or the failure mode of plastic collapse exceeds M_y and is α times M_y . Therefore, a structural element can withstand the bending moment above the first yield point bending moment, and its factor to represent the extension of the allowable range of elastically calculated stress intensity is the shape factor α . Accordingly, incorporating the design factor, the bending stress intensity P_b is limited to less than or equal to αS_m under pure bending.

In summary, in the current design codes, allowable stresses have an adequate margin of load-carrying capacity (called the effect of shape factor) for a bending moment. In Table 5.10, the membrane plus bending stress is 1.5 times the design stress intensity S_m and therefore, can be understood 1.5 times the general membrane stress.

In the field of architectural engineering and the related fields, the shape factor is defined as $\alpha = Z_p / Z$, where Z is the section modulus defined in Sect 2.3.4 in Chap. 2, and Z_p is called the plastic section modulus. By using Z_p , M_c can be expressed as $M_c = S_y Z_p$, where S_y is the yield strength. It is clear that the expression $\alpha = Z_p / Z$ is equivalent to Eq. (5.11).

Specific values of shape factor are as follows: $\alpha = 1.5$ for a rectangular section bar, $\alpha = 1.27$ for a cylindrical pipe, $\alpha = 1.10$ – 1.18 for an I-section beam and, $\alpha = 1.70$ for a cylindrical bar.

[3] Limits on local primary stress

Local primary membrane stress is developed by an external load at a structural discontinuity. Its value depends on how the structure is constrained. It is self-limiting unlike general primary membrane stress in a general region, which is characteristic of secondary stress. If, for example, local membrane stress or membrane stress at a structural discontinuity due to an external load exceeds the yield strength, other regions are affected and the stress distribution in these regions changes to resist the load. As a result, redistribution of stress occurs.

Due to this shift in stress distribution, the distortion resulting from the shift can be unacceptably large. In the current structural codes, the stress in this case is treated as primary stress. Therefore, the membrane stress at a structural discontinuity due to an external load is treated as local primary membrane stress P_L . The stress is limited to below S_y , and the stress limit is set to $1.5S_m$.

[4] Summary of primary stress limits (primary stress (intensity) limits given in structural codes)

Table 5.10 summarizes the above considerations.

Referring to Ref. [2], the following provides essential requirements as the stress limits for other than bolts of class 1 vessel for design loadings and level A-D service loadings.

(i) Design loadings (Referring to Ref. [2], NB-3221)

(a) General primary membrane stress intensity (NB-3221.1)

The allowable value of this stress intensity P_m produced by design internal pressure (as repeated mentioned, in Japanese codes like Ref. [1], the design internal pressure is denoted as the maximum allowable working internal pressure.) and other specified design mechanical loads (but excluding all secondary and peak stress) is S_m at the design temperature, that is to say, the following expression.

$$P_m \leq S_m \quad (5.12)$$

Membrane stress intensity is derived from the stress component averaged across the thickness of the section. The averaging shall be performed at the stress component level.

(b) Local membrane stress intensity (NB-3221.2)

The allowable value of this local primary membrane stress intensity P_L produced by design pressure and specified design mechanical loads (but excluding all thermal and peak stresses) is $1.5S_m$.

$$P_L \leq 1.5S_m \quad (5.13)$$

The intention to define the local primary membrane stress intensity is described in Item [3] above.

A stressed region may be considered local if the distance over which the membrane stress intensity exceeds $1.1S_m$ does not extend in the meridional direction more than $1.0\sqrt{Rt}$ (R is the minimum midsurface radius of curvature and t is the minimum thickness in the region considered). Regions of local primary stress intensity involving axisymmetric membrane stress distributions which exceed $1.1S_m$ shall not be closer in the meridional direction than $2, 5\sqrt{Rt}$ ($R = (R_1 + R_2)/2$, $t = (t_1 + t_2)/2$, where R_1 and R_2 are the minimum midsurface radii of curvature at these regions where the membrane stress intensity $1.1S_m$, and t_1 and t_2 are the minimum thickness at each of the regions considered).

The definition of "local" is given in NB-3213.10.

- (c) Primary membrane (general or local) plus bending stress intensity (NB-3221.3)

This stress intensity is derived from the highest value across the thickness of a section of the general or local primary membrane stresses plus primary bending stresses produced by design pressure and other specified design mechanical loads, but excluding all secondary and peak stresses. For solid rectangular sections, the allowable value of this stress intensity is $1.5S_m$. For other than solid rectangular sections, a value of α times the limit established in Item a) above may be used, where the factor α is defined as the ratio of the load set producing a fully plastic section to the load set producing initial yielding in the extreme fibers of the section. The value of α shall not exceed the value calculated for bending only ($P_m = 0$). In no case shall the value of α exceed 1.5.

Thus, the allowable value of the primary membrane (general or local) plus primary bending stress intensity can be expressed as follows.

$$P_L + P_b \leq 1.5S_m \quad (5.14)$$

- (ii) Levels A and B service limits

An interpretation of this limit is given in Sect. 5.9.2.

There are no specific limits established on the primary stresses in the Level A limits, because the primary stress intensity limits are required to be satisfied for the design loadings. The design loadings shall not be less than the maximum loadings, which exist under the most severe loadings for which the Level A service limits are applicable. The Level B service loadings may be taken into account in determining the design loadings, although as the Code rules (NB-3223 (a) (1)), for primary stress intensities generated by Level B service loadings, allowable stress intensity values of 110% of those given in Item (i) above shall apply.

As mentioned above, all events or all representative events of categorized classes shall be basically evaluated when the Design by Analysis approach is applied to nuclear reactor components.

It can be understood that the representative events categorized as the service condition A are encompassed by those of the design condition, and correspondingly, those of the service condition B are taken into account in determining the design loadings. This may be due primarily to a historical background of the Design by Analysis approach. At that time when the Design by Analysis approach was established, there was no application experience of this approach. A concept of sizing that has been established as the Design by Formula approach (roughly speaking, sizing for the primary stresses under the design condition), was said to be introduced to enhance a design reliability.

- (iii) Level C service limits (NB-3224.1)

The primary stress limits of Item (i) above shall be satisfied using an S_m value equal to the greater of 120% of the tabulated S_m value or 100% of the

tabulated yield strength, with both values taken at the appropriate temperature, $\max.(1.2S_m, 1.0S_y)$, as expressed below:

- (a) General primary membrane stress intensity;

$$P_m \leq \max.(1.2S_m, 1.0S_y) \quad (5.15)$$

- (b) Local primary membrane stress intensity;

$$P_L \leq 1.5 \times \max.(1.2S_m, 1.0S_y) \quad (5.16)$$

- (c) Primary membrane (general or local) plus primary bending stress intensity;

$$P_L + P_b \leq 1.5 \times \max.(1.2S_m, 1.0S_y) \quad (5.17)$$

For other than solid rectangular sections, an allowable value of α times the limit established in the above expression, i.e., $\alpha \times \max.(1.2S_m, 1.0S_y)$ may be used. The value of α shall not exceed the value calculated for bending only ($P_m = 0$). In no case shall the value of α exceed 1.5.

In addition, for ferritic materials, the P_m elastic analysis limits for pressure loadings alone shall be equal to the greater of $1.1S_m$ or $0.9S_y$, i.e., $\max.(1.1S_m, 0.9S_y)$.

- (iv) Level D service limits (NB-3225)

Rules contained in Appendix F may be used in evaluating level D service loadings (service loadings for which level D limits are designated), independently of all other design and service loadings.

Appendix F is the ASME Boiler and Pressure Vessel Code Section III Division 1 Appendices, Nonmandatory Appendix F "Article F-1000 Rules for evaluation of service loadings with level D service limits." In Appendix F-1000, the level D service limits and design are provided for limiting the consequences of the specified events. They are intended to assure that violation of the pressure-retaining boundary will not occur, but are not intended to assure operability of components either during or following the specified event. System analysis is performed to determine loads on component and supports. A system is an assemblage of components, supports, and other interconnected structures. System analysis may be performed by elastic analysis methods or by plastic analysis methods. The acceptance criteria in which only limits on primary stresses are prescribed are as follows, when elastic system analysis is used to determine loads on components.

- (a) General primary membrane stress intensity;

for austenitic steel, high-nickel alloy, and copper-nickel alloy materials,

$$P_m \leq \min .(2.4S_m, 0.7S_u) \quad (5.18)$$

for ferritic steel materials,

$$P_m \leq 0.7S_u \quad (5.19)$$

- (b) Local primary membrane stress intensity;

P_L shall not exceed 150% of the limit for general primary membrane stress intensity P_m , i.e., for austenitic steel, high-nickel alloy, and copper-nickel alloy materials,

$$P_m \leq 1.5 \times \min .(2.4S_m, 0.7S_u) \quad (5.20)$$

for ferritic steel materials,

$$P_m \leq 1.5 \times 0.7S_u \quad (5.21)$$

- (c) Primary membrane (general or local) plus primary bending stress intensity;

$P_L + P_b$ shall be limited in accordance with the one of the following provisions;

- stress limits below, and
- Item (d) below.

Stress limits; stress intensity $P_L + P_b$ shall not exceed 150% of the limit for general primary membrane stress intensity P_m , i.e., for austenitic steel, high-nickel alloy, and copper-nickel alloy materials,

$$P_L + P_b \leq 1.5 \times \min .(2.4S_m, 0.7S_u) \quad (5.22)$$

for ferritic steel materials,

$$P_L + P_b \leq 1.5 \times 0.7S_u \quad (5.23)$$

- (d) Collapse load

Static or equivalent static loads shall not exceed 90% of the limit analysis collapse load using a yield stress which is lesser of $2.3S_m$ and $0.7S_u$, i.e., $\min .(2.3S_m, 0.7S_u)$, or 100% of the plastic analysis collapse load or test collapse load. Here, note that limit analysis collapse load is defined in NB-3213.28 in Ref. [2], in more detailed in Item [1] in Section 5.8.2, as follows;

Applying a limit analysis, at this load, the deformations of the structure increase without bound, because as described in Item [2] 3) above, the fully plastic section is formed at this load. Similarly, a plastic analysis-collapse

load is defined in NB-3213.25, and is described in more detail in Item [2] in Section 5.8.2. If this method is used, particular care should be given to ensure that the strains or deflections that are used are indicative of the load-carrying capacity of the structure.

It should be mentioned that in the above, the factor multiplying to S_u for deriving the allowable stress is 0.7 for level D service limits, while in a Japanese code or standard for nuclear facility components such as Ref. [1], this value is 2/3, as shown in Table 5.12 in Sect. 5.9.2. This difference in S_u factor seems due to different historical backgrounds in the process of developing technical standards between in U.S. and Japan.

5.6.2 *Primary Plus Secondary Stress Limits*

[1] Basic concept

A single or cyclic application of secondary stress alone does not practically cause ductile fracture and does not accumulate an excessive deformation, as shown in Fig. 5.2. However, even though the primary stress is limited less than the yield strength, some applications of a certain range of combined primary stress and secondary stress may develop a plastic deformation, and there is likelihood of accumulating an excessive deformation, as a result of progressive (or incremental) plastic deformation across the entire wall thickness of a structural element every stress cycle.

Hence, primary plus secondary stress is limited for the following reasons:

- (1) Prevent incremental collapse or loss of function due to incremental deformation resulting from excessive plastic deformation under cyclic loading.
- (2) In the current structural design codes, the usage factor is evaluated using the stress from elastic analysis. Therefore, the stress limit is placed to validate fatigue evaluation based on elastic analysis.

The best possible way to meet the above two requirements is to demonstrate that the structure is elastically deformed (this is called elastic response) under cyclic loading. To be exact, the range of primary plus secondary stress ($P + Q$) is limited to allow the elastic response of the structure. If the range of ($P + Q$) exceeds the yield strength S_y , permanent plastic deformation occurs. It is known that for ($P + Q$) within a certain range, the structure is plastically deformed only in the first few cycles and subsequently exhibits elastic response. This structural behavior is called “shakedown to elastic action” or simply “shakedown.” In a case of an actual work-hardening material, after the first few cycles, the shakedown may occur, while in a case of elastic-perfectly plastic material, the structure may be plastically deformed only at the first cycle and subsequently may exhibit elastic response.

First, we consider a cyclic application of thermal stress alone. The simplest case was shown in Fig. 5.2. In this case, the bar is not initially constrained, that is the top end of the bar is not in contact with a wall. After the top end reaches the wall during the

heating stage, a compressive plastic deformation is developed. On the assumption of elastic-perfectly plastic material, at the heating stages of the second and subsequent cycles, it is just in contact with the wall, but the further plastic deformation is no longer developed, that is the shakedown occurs.

Next, we consider another simple model to exhibit a little different structural response from the above case. The model is shown in Fig. 5.14a, b. A bar with the initial length l_0 , constrained at the top and lower ends is subjected to thermal cycles. In the thermal cycle, the bar is cooled from room temperature and then returned to room temperature. The bar is cooled to develop a tensile thermal stress at the former stage (or loading stage) of the first cycle and the subsequent cycles. Figure 5.14d shows the stress-strain relationship of the bar of elastic-perfectly plastic material. Let S_1 be the stress when cooled below room temperature, obtained from elastic calculation (shown by the dashed line in Fig. 5.14d (i)). Then, the response of the elastic-perfectly plastic material for $S_y < S_1 < 2S_y$ is as shown by the solid line in Fig. 5.14d (i). In the first loading cycle, the bar yields when point A is reached during the cooling process, and subsequently, the stress remains constant at S_y , eventually reaching point B. The stress changes from 0 to A to B, resulting in the plastic deformation as indicated as $\overline{AB}$. In this figure, the strain is the sum of elastic strain plus plastic strain excluding thermal strain. Compatibility condition of strain may be expressed as follows: thermal strain ($\alpha\Delta T$) + elastic strain $\dot{\epsilon}_e(=\sigma/E)$ + plastic strain = 0, because the length of the bar is always constrained to the initial length l_0 . During unloading, the stress decreases from point B in parallel with 0A as shown in the figure. Since the bar is constrained to the initial length, the stress returns to point C with zero strain as undeflected position. As a result, residual stress with the magnitude indicated by 0C is developed. In the second cycle, the stress starts at point C and reaches point B under loading. However, due to $S_1 < 2S_y$, the stress remains at point B.

Subsequently, this pattern is repeated and no yielding occurs. In the first loading cycle, the bar yields and is subjected to plastic strain, but, due to the resulting residual strain, it exhibits elastic response after the first unloading. The phenomenon in which subsequent structural response is elastic after plastic deformation is called shakedown, as mentioned above.

For $S_1 > 2S_y$ as shown in Fig. 5.14d (ii), no shakedown occurs. During the first unloading, the stress changes from D to E to F and reaches the compressive yield stress. In the second loading cycle, the stress starts at point F or the stress $-S_y$ and increases in parallel with 0A again, reaching the tensile yield stress. Due to $S_1 > 2S_y$, the stress tries to increase, but cannot increase due to the elastic-perfectly plastic material. However, as cooling continues, the bar tries to further contract and therefore, tensile elongation is required to keep it at the initial length. Then, the stress reaches point D. Subsequently, the cycle of tensile yielding and compressive yielding is repeated. The important point is that in this case, plastic deformation is not accumulated in each cycle. This is because the positive plastic strain due to the tensile yield is offset by the compressive yield. As a result, a hysteresis loop D-E-F is formed as shown in Fig. 5.14d (ii). The response in which deformation is not accumulated but the cycle of tensile and compressive yielding is repeated is called

plastic cycling and represented by the symbol P . In a single bar, no further behavior is observed. In a pair of bars shown in Fig. 3.29 (this model is called “two-bar structure”) and a real structure such as a cylinder, a cyclic application of thermal cycle under constant primary stress may cause accumulation of plastic deformation in both bars of the two-bar structure and in the real structure, across the entire wall thickness, resulting in an incremental plastic deformation. This type of structural response is called “ratchetting” or “ratchet.” The detail is described in Item [3] below.

In order to satisfy the requirement (1) above, it is possible to allow the accumulation of plastic deformation due to ratchetting under the limitation of the accumulated plastic deformation below the allowable limit so as to maintain required functions, as well as to limit the allowable range of combined stresses within the elastic and shake-down regimes. However, several issues should be resolved for developing plastic analysis methods that are applicable to ratchetting response under the cyclic application of combined primary and secondary stresses. At present, such methods have been developed for several steel materials such as austenitic stainless materials.

The small plastic strain developed in the first few cycles is not harmful under the allowable range of primary stress that is limited to the elastic regime. Therefore, demonstrating that shakedown occurs but the ratchetting does not is used as the criterion to meet the requirements (1) and (2) above. In Fig. 5.14, the shakedown boundary is $2S_y$.

From the above discussion, the basic criterion for achieving the requirements (1) and (2) is to limit the range of $(P + Q)$ to below $2S_y$.

[2] $P + Q \leq 3S_m$

As described above, $P + Q \leq 2S_y$ is required for a structure to shake down so that it exhibits elastic response. However, in the current structural codes, the value for as-received material is used as S_y . For cyclic softening materials, whose yield strength decreases as cycling with inducing plastic deformation, the use of the value for as-received material as the design yield strength is not conservative.

For these materials, the design stress intensity S_m as the allowable stress is determined by the design tensile strength instead of the design yield strength, because a yield ratio (defined as the yield strength/tensile strength) exceeds 0.5. That is, $S_m = (1/3)S_u$. Based on an engineering judgment, $3S_m$, instead of $2S_y$, is used as the allowable range of $(P + Q)$ for a structure to shake down, which is given by

$$P + Q \leq 3S_m \quad (5.24)$$

In the equation, the multiplication factor of 3 is used for S_m because $S_m = \min.(2/3 \times S_y, 1/3 \times S_u)$, or $3S_m = \min.(2S_y, S_u)$, and then, $3S_m \leq 2S_y$.

Conversely, $P + Q \leq 2S_y$ is reasonable for cyclic-hardening materials. In the current structural codes, there is no limitation on the materials and therefore $3S_m$ is used instead of $2S_y$.

[3] In the case of $P + Q \geq 3S_m$ (Part 1): when no shakedown is allowed (thermal ratchet limits) (i.e., allowing plastic cycling)

The above-mentioned shakedown criterion, $P + Q \leq 3S_m$, for limiting cyclic loadings excludes plastic cycling and therefore is unduly conservative. This is because plastic cycling does not involve the accumulation of plastic strain and therefore is allowed for the prevention of progressive deformation. However, fatigue evaluation on cyclic plastic straining must be performed, because cyclic plastic straining with alternating tensile and compressive plastic strains occurs in both areas of outer surface and inner surface, where a high stress exceeding the yield strength is induced. For the prevention of progressive deformation, it is reasonable to properly evaluate the response of the structure and allow the deformation within a range that does not lead to ratcheting.

With this background, Bree [11] and Miller [12] obtained the deformation behavior of a typical structure subjected to sustained pressure and other mechanical loadings under cyclic thermal loadings. They analyzed the response of an axisymmetric cylinder steadily subjected to primary stress such as pressure under cyclic thermal stress conditions, based on the conservative assumption that the material is elastic-perfectly plastic, and then classified the structural response into a response diagram. Most pressure-retaining components such as boilers and pressure vessels are steadily subjected to normal operating pressure and additionally subjected to thermal changes or fluctuations during normal startup and shutdown and abnormal shutdown. Thermal stress cycles are applied in addition to the sustained primary stress due to pressure and other loads. Originally, this was an issue in structural response analysis of aircraft wings subjected to aerodynamic heating, and developed into an issue with fuel cladding.

Figure 5.15 shows the analytical results of a cylinder with a linear temperature distribution across the wall thickness due to through-thickness thermal gradient. The response of the cylinder is classified in the figure. In addition, the design window consisting of the previously mentioned shakedown limit and the design window of the primary stress intensity is given in this figure. The shakedown limit is expressed as $P + Q \leq 3S_m$. Here, it is expressed as $P + Q \leq 2S_y$. Also, the primary intensity limit is expressed as $P_m \leq 2/3S_y$.

Diagrams developed by Bree or Miller are illustrated as a function of circumferential or axial thermal stress as the thermal stress. It is a significant stress (the highest component of thermal stress) but not an equivalent stress, that is stress intensity. Since design diagrams for preventing incremental collapse or ratchetting as shown in Fig. 5.15 are based on the diagrams by Bree or Miller, stresses to be evaluated are, therefore, the thermal stress component in the circumferential or axial direction, which is the higher, and the general primary membrane stress component in the direction that corresponds to the direction of the thermal stress. In other words, general primary membrane stress σ_m is considered for the primary stress P and thermal stress σ_t is considered for the secondary stress Q .

From this figure, the following features may be pointed out.

- (i) The figure shows that the range of stress that causes the shakedown of a structure such as a cylinder (in the figure, S_1 and S_2) is wider than the design window $2S_y$.

(ii) It also shows the stress range for plastic cycling and thermal ratcheting.

A detailed analysis shows that there are two major patterns in the response in the form of shakedown and ratcheting, according to the combination of general primary membrane and thermal stresses: regimes S_1 and S_2 for shakedown and regimes R_1 and R_2 for ratcheting. Figure 5.16 conceptually shows the change in stress distribution across the thickness in each regime. In the figure, the shaded part of the area where the stress is equal to the yield strength S_y is the newly yielded region (yielding may not always occur when the stress reaches the yield stress). The patterns are due to the difference in how yielding occurs as described below:

- Only tensile yielding (the regimes are S_1 and R_1)
Only tensile yielding occurs due to a high general primary membrane stress
- Tensile yielding on one side and compressive yielding on the other side (the regimes are S_2 and R_2)

Yielding also occurs on the compression side due to a combination of low general primary membrane stress and a high thermal stress. In this case, tensile and compressive yielding occurs alternately on the surface, showing the plastic cycling response. For R_2 , yielding occurs even in the core region (in the central region), and therefore, through-thickness accumulation of plastic deformation is developed.

The boundaries between these regimes are given by the following equations.

The boundary between the elastic regime E and the shakedown regime S_1 :

$$\sigma_m + \sigma_t = S_y \quad (5.25)$$

The boundary between the shakedown regimes S_1 and S_2 , and between the ratcheting regimes R_1 and R_2 :

$$(S_y - \sigma_m)\sigma_t = S_y^2 \quad (5.26)$$

The boundary between the shakedown regime S_2 and the plastic cycling regime P :

$$\sigma_t = 2S_y \quad (5.27)$$

The boundary between the plastic cycling regime P and the ratcheting regime R_2 :

$$\sigma_t \sigma_m = S_y^2 \quad (5.28)$$

The boundary between the shakedown regime S_1 and the ratcheting regime R_1 :

$$\sigma_m + \frac{\sigma_t}{4} = S_y \quad (5.29)$$

Now, we discuss the reason why the range of stress that causes the shakedown of a structure such as a cylinder is wider than the design window for $P + Q \leq 2S_y$. We consider a case of the regime S_1 in which the general primary membrane stress is high and only tensile yielding occurs. Figure 5.17a shows the cylinder model and (b) shows the thermal cycles considered here. The cylinder has an infinite length and a large diameter compared to the thickness. Its stress state can be approximated by that of a flat plate. Then, the elongation in the circumferential direction is uniform across the wall thickness (this is called uniform elongation). We focus on the circumferential direction and assume an elastic-perfectly plastic material. Figure 5.17c shows the stress distribution from an elastic calculation when the temperature difference between the inner and outer surfaces of the cylinder reaches the maximum value denoted as ΔT .

For $\sigma_m + \sigma_t \geq S_y$, tensile yielding occurs on the outer surface. The stress in the elastic-perfectly plastic material does not exceed the design yield point S_y , and therefore a redistribution of stress occurs as described in the discussion of a bending moment. This stress distribution must meet the equations of the equilibrium for the force and the moment given by

$$\int_{-t/2}^{+t/2} \sigma dx = \sigma_m t \quad (5.30)$$

$$\int_{-t/2}^{+t/2} \sigma x dx = 0 \quad (5.31)$$

where t is the wall thickness.

In Eq. (5.30), the stress distribution is required to be in equilibrium with the internal pressure p applied as the external load. Then, $\sigma_m = Dp/2t$. It should be noted that the above equations are not met by the distribution curve shown by the dot-and-dash line in Fig. 5.17d(i) on the assumption of elastic-perfectly plastic material, which is obtained by replacing the part of the stress distribution curve from the elastic calculation that exceeds S_y with S_y . A slope of stress distribution meeting the equilibrium condition, particularly Eq. (5.30), shall be the slope of elastically calculated thermal stress distribution, and therefore, must coincide with that of the dot-and-dash line. As a result, the stress distribution to meet the equilibrium condition must be as shown by the solid curve. The yielded region is deeper toward the center of the plate than that obtained by the elastic calculation. The coordinate value a showing the yielded region is given by

$$a = \frac{-t + 2t\sqrt{(S_y - \sigma_m)/\sigma_t}}{2} \quad (5.32)$$

Figure 5.17d(ii) shows the distribution of plastic strain. As described later, we have $a > 0$ for $(S_y - \sigma_m)/\sigma_t > 1/4$, or $\sigma_m + (1/4)\sigma_t < S_y$. This is the criterion for the yielded region not to extend beyond the center of the plate and one of the criteria for

shakedown. During unloading in the first cycle, the temperature distribution returns to the distribution in the initial state, leaving plastic strain. However, the elongation across the thickness must be uniform and therefore stress is developed to offset the residual plastic strain. In other words, residual stress is developed. The dashed curve in Fig. 5.17e is the distribution of the residual stress obtained from the elastic calculation that is developed to offset the residual plastic strain.

Unfortunately, a stress distribution in which a uniform stress distribution at σ_m is added to the above does not meet the force equilibrium condition. The solid curve is the correct stress distribution to meet the equilibrium condition. The stress σ^* in the flat part of the curve is given by

$$\sigma^* = S_y + \sigma_t - 2\sqrt{\sigma_t(S_y - \sigma_m)} \quad (5.33)$$

If $\sigma^* \geq S_y$, tensile yielding occurs for $x \leq a$. Then, we have $\sigma_m + (1/4)\sigma_t < S_y$. In this case, positive plastic strain is accumulated through thickness of the plate in the first cycle. If $\sigma^* \geq S_y$, plastic strain is accumulated through thickness in each cycle after the first cycle. Ratcheting occurs. As a result, the range of stress in the ratcheting regime R_1 is given by $\sigma^* \geq S_y$, and then

$$\sigma_m + (1/4)\sigma_t \geq S_y \quad (5.34)$$

This criterion is also $a \leq 0$. You may want to examine the criteria for R_2 .

As you can see from the above discussion and Fig. 5.16, the material shakes down if the center portion of the plate wall, i.e., the core region, remains in the elastic regime throughout the cycle, and ratchets if tensile yielding occurs. If the material shakes down, the center portion always remains in the elastic regime, and in this chapter, this center portion remaining in the elastic regime is called the “elastic core”.

Now, return to the discussion on wider stress range to cause the shakedown than the design window, that is $P + Q \leq 3S_m$, or $\sigma_m + \sigma_t \leq 2S_y$. Here, investigate what kind of stress distribution is produced in the stress range: $\sigma_m + \sigma_t \leq 2S_y$, in particular, on the boundary: $\sigma_m + \sigma_t = 2S_y$. Substituting this equation into Eq. (5.33) leads to

$$\sigma^* = S_y + \sigma_t - 2\sqrt{\sigma_t(\sigma_t - S_y)}.$$

As $\sigma_t - S_y < \sigma_t$,

$$\sigma^* > S_y + \sigma_t - 2\sqrt{\sigma_t \times \sigma_t} < S_y \quad (5.35)$$

From the above investigation and Fig. 5.15, σ^* may be below S_y on the boundary $\sigma_m + \sigma_t = 2S_y$. This means that the boundary may be within the shakedown regime. We can conclude that the generic shakedown limit $\sigma_m + \sigma_t \leq 2S_y$ or $P + Q \leq 3S_m$ may be conservative as the shakedown limit for a cyclic application of thermal stress under a constant primary stress acting on cylindrical shells.

The stress limits for preventing thermally induced ratchetting may be called thermal ratchetting limits.

In current high-temperature structural design codes applicable to both the non-creep and creep regions, the allowable range is limited to the structural response regimes in which ratcheting does not occur, namely, the elastic behavior regime E , the shakedown regimes S_1 and S_2 , and the plastic cycling regime P .

[4] In the case of $P + Q \geq 3S_m$ (Part 2): evaluating the increments of fatigue damage (K_e factor)

Thermal ratcheting limits are established for cyclic thermal stress. Allowable structural responses include elastic behavior, shakedown, and plastic cycling. As discussed above, the generic shakedown limit $P + Q \leq 3S_m$ excludes the stress range in which the plastic cycling occurs. If the thermal ratchetting limit is applied as a design rule, instead of the generic shakedown limit, plastic strains are developed on the inner and outer sides in the stress range of plastic cycling. Since an alternating plastic strain cycling causes fatigue damage, a fatigue damage evaluation is required for the stress range of plastic cycling. Further, a larger fatigue damage may be induced for a stress range beyond the above-mentioned design window: $P + Q \leq 3S_m$ than that evaluated using an elastic analysis method.

Areas of stress or strain concentration such as structural discontinuities are susceptible to fatigue damage. Loading at structural discontinuities is strain-controlled, and therefore the prediction of strain is important. If $P + Q \leq 3S_m$ or $P + Q \leq 2S_y$, the strain range can be calculated by elastic analysis. Even if significant plasticity is present at a structural discontinuity, peak strain and stress can be calculated, using the strain and stress concentration factors, as long as the nominal strain or stress is below the limit of $2S_y$. If $2S_y$ is exceeded, the plastic strain concentration at the structural discontinuity is enhanced, making it difficult to calculate peak strain and stress using the strain obtained by elastic analysis. If $2S_y$ is exceeded, the following factors should be considered in fatigue evaluation:

- Peak plastic strain calculation
- Plastic redistribution of nominal strain resulted from expanded plasticity (particularly where there is elastic follow-up)
- Ratcheting

It is not practical to accurately consider these factors for each structural configuration due to extensive analysis required. Therefore, in the current design codes, an approach is used in which the effect of important factors in evaluation on simple configurations is evaluated and generalized. As a result, the stress amplitude S_a on the vertical axis of the design fatigue curve is given by the following equation in terms of the alternating stress intensity S_{alt} obtained by elastic analysis [13]:

$$\begin{aligned}
 S_a &= K_e \cdot S_{alt} \\
 K_e &= 1.0 \quad S_n \leq 3S_m \\
 &= 1.0 + [(1 - n)/n(m - 1)](S_n/3S_m - 1) \quad 3S_m < S_n < 3mS_m \\
 &= 1/n \quad S_n \geq 3mS_m
 \end{aligned} \tag{5.36}$$

where S_n : range of primary plus secondary stress intensity ($= P_L + P_b + Q$)

m, n : material constants

The following requirements must be met:

- (1) The range of the primary plus secondary stress intensity obtained excluding thermal bending stress must be below $3S_m$.
- (2) The thermal ratcheting limit must be met.
- (3) The yield ratio (the ratio of the specified minimum yield strength to the specified minimum tensile strength, as the terminology of the ASME Boiler and Pressure Vessel Code) must be below 0.8. This requirement is intended to exclude low-ductility materials.

Recently, problems (for instance, the above equation is unduly conservative) were identified with the above equation, and improvements were proposed. The improvements were incorporated in some design codes.

5.6.3 Total Stress Limits, Including Peak Stress (Fatigue Damage Limits)

[1] Basic concept

The purpose of fatigue evaluation in the design codes is to evaluate the fatigue performance of components but not to predict fatigue damage that may occur under actual operating conditions. It is difficult to predict actual operating conditions in the design stage. Therefore, fatigue performance is evaluated on the basis of conservatively postulated operating conditions for design.

The Design by Analysis approach requires fatigue evaluation to justify the reduction of design factors, for example, for S_u from 4 to 3. The number of stress cycles anticipated during the design life of the pressure vessel and piping in a nuclear power plant rarely exceeds 10^5 , in particular, for thermal fatigue. Therefore, low-cycle fatigue evaluation became important in fatigue evaluation. Before then, fatigue evaluation had been at best based on endurance limits, so-called fatigue limit design.

The principle of fatigue evaluation remains unchanged since it was included in the requirements in the ASME Boiler and Pressure Vessel Code Section III Division 1 Subsection NB, Class 1 Components in 1963. It is briefly described below:

- Fatigue damage due to cyclic loading is accumulated.
- Fatigue damage is estimated using the usage factor U given by

$$U = \frac{n}{N_f} \quad (5.37)$$

where n is the number of stress cycles and N_f is the allowable number of cycles (obtained from the design fatigue curve) at the stress amplitude of the cycle (or

alternating stress intensity). A method for generating the design fatigue curve is provided in Item [2] below.

- $U \leq 1.0$
- When multiple stress cycles with different stress amplitudes are applied, fatigue damage is evaluated using the linear summation of the partial usage factors given by

$$U = \sum_i U_i = \sum_i \frac{n_i}{N_{f,i}} \quad (5.38)$$

The above expression seems rather simple. Here, we have to remind that a real component undergoes complicated stress or strain cycles, whereas a fatigue testing specimen undergoes very simple or idealized stress or strain cycles (typically, a sine-wave strain-controlled fatigue testing with a constant strain amplitude), as is explained in Figs. 5.23 and 5.24. It is not easy to select stress or strain cycles with constant amplitudes from real stress or strain cycles, for applying the selected cycles to the design fatigue curve. This issue is discussed in Item [2] (3) below.

The above three statements mean that a damage of $1/N_f$ is caused to the material in each stress cycle and accumulated, and that when the fractions of damage linearly add up to 1.0, failure occurs. Because of this, Eq. (5.38) is called the linear damage rule.

- As shown below, the alternating stress intensity S_{alt} for determining the allowable number of stress cycles is given by 1/2 of the absolute value of the range of each principal stress difference $|S_{ij, \max} - S_{ij, \min}|$, as with Tresca stress intensity, given by

$$S_{ij} = \sigma_i - \sigma_j \quad (5.39)$$

That is, S_{alt} is given by the following equation in terms of the principal stress difference S :

$$S_{alt} = \frac{1}{2}(S_{\max} - S_{\min}) \quad (5.40)$$

$S_{\max}$ is the maximum in the stress cycle, as illustrated in Fig. 5.18, and $S_{\min}$ is the minimum.

- Fatigue evaluation focuses on normal service conditions (service conditions A and B), because a significant fatigue damage is caused by cyclic applications of stress or strain cycles.
- The stress to be evaluated is the total stress including peak stress. Design fatigue curves are used to estimate the allowable number of cycles (or the maximum number of repetitions) as a function of the alternating stress intensity S_{alt} , as defined above, one-half of the alternating stress intensity range. It should be noted here that peak stress is an increment of stress which is additive to the primary

plus secondary stresses by reason of local discontinuities or local thermal stress including the effects, if any, of stress concentrations (Ref. [2], NB-3213.11). On the other hand, relating to the alternating stress intensity, peak stress intensity is derived from the highest value at any point across the thickness of a section of the combination of all primary, secondary, and peak stresses (Ref. [2], NB-3222.4 (b)).

[2] Generating design fatigue curves and the effect of mean stress

(1) Generating design fatigue curves

As mentioned above, evaluation of low-cycle fatigue is important in the design of pressure vessels. In low-cycle fatigue, the stress in the vessel exceeds the yield stress and plastic strain occurs. In the plastic region, a small increase in stress causes a large increase in plastic strain. It is known that fatigue damage in the plastic region due to cyclic loadings can be expressed as a function of plastic strain. Therefore, a fatigue test focusing on the plastic region should be performed under strain-controlled loading, instead of stress-controlled loading, which has traditionally been used in a high-cycle fatigue test. The number of cycles to failure should be correlated to the range of strain. In design for fatigue, a virtual elastically calculated stress obtained by multiplying strain, a control variable for a fatigue test, by Young's modulus is used to make it easier to directly use the results of elastic analysis. For this reason, a design fatigue curve is called an *S-N* diagram (or curve).

A strain-controlled fatigue test is performed to generate *S-N* curves. Strain is converted to elastically calculated stress using the fatigue test data to obtain best-fit curves (regression curves). In the ASME Boiler and Pressure Vessel Code, the following equation had been used to calculate the virtual elastically calculated stress [14] *S*:

$$S = \frac{E}{4\sqrt{N_f}} \ln \frac{100}{100 - A} + B \quad (5.41)$$

where *E*: Young's modulus

N_f: number of cycles to fatigue fracture (defined by the 25% descent method), as is described in Item (3) below, or in more detail, (a) in Figure 5.23

S = total strain amplitude × Young's modulus

A, B: material constants

In some cases, the percent reduction of area in a tensile test is used as *A* and the endurance limit is used as *B*. The above equation is simple but has some drawbacks, in particular, in performing an accurate calculation. Methods with improved accuracy were proposed.

A design fatigue curve is obtained taking into account design factors of 2 and 20 in the thus obtained best-fit curve, as described in Item [2] in Sect. 5.3.2. These design factors is thought to enable derivation of number of cycles to fatigue crack initiation from the best-fit fatigue fracture curve. However, if the effect of mean stress

is expected, the best-fit curve for high-cycle fatigue is modified taking into account the effect of mean stress. And then, the design fatigue curve is derived from the modified best-fit curve.

(2) The Effect of mean stress

It is known that if the stress cycles are not fully alternating cycles and a mean stress component is present, the life for high-cycle fatigue (the number of cycles to fracture) where the stress amplitude is below the level of the yield stress decreases.

For example, the stresses in the stress cycle (from elastic analysis) in Fig. 5.18 are given by

$\sigma_{\max}$: algebraic maximum in the stress cycle

$\sigma_{\min}$: algebraic minimum in the stress cycle

$\sigma_{\text{alt}} = \frac{1}{2}(\sigma_{\max} - \sigma_{\min})$: Alternating stress amplitude component

$\sigma_{\text{mean}} = \frac{1}{2}(\sigma_{\max} + \sigma_{\min})$: Mean stress component

This shows the results of elastic analysis. When the maximum stress exceeds the yield stress, plastic strain occurs and, due to the resulting residual stress, the actual stress components are different from those from elastic analysis. Particularly when the stress range exceeds $2S_y$, plastic cycling occurs and the mean stress component becomes zero after a few cycles, eliminating the need to consider the effect of mean stress. Hereinafter, the component is omitted.

Now, we examine how the effect of mean stress is incorporated into the design fatigue curve when the stress amplitude is below the yield strength. First, we examine how the stress cycle changes after plastic deformation occurs.

To be conservative, we assume an elastic-perfectly plastic material. If the maximum stress from elastic analysis $\sigma_{\max} = \sigma_{\text{mean}} + \sigma_{\text{alt}}$ exceeds the yield strength, as shown in Fig. 5.19b, the stress in the elastic-perfectly plastic material goes through point A, moving on the horizontal line to point B, because the stress cannot exceed the yield strength. Next, during the unloading process, the stress decreases and goes through the point C, reaching point D at the end of this unloading process. The stress at point D is equal to $S_y - 2\sigma_{\text{alt}}$, and therefore, the compressive yielding does not occur. In this case, an elastic action only is possible throughout one cycle from the second cycle onward. As mentioned above, this response is called “shakedown.” For this type of stress response, the mean stress after shakedown is no longer the elastically calculated mean stress in the first cycle, σ_{mean} , but is shifted to the mean stress denoted by point C. In a case of a real work-hardening material, the shakedown occurs after several stress cycles, and correspondingly, the mean stress is shifted. When the elastically calculated stress range exceeds $2S_y$, as shown in Fig. 5.19c, from the second cycle onward, the stress cycle in the elastic-perfectly plastic material changes to form a hysteresis loop $B \Rightarrow C \Rightarrow D \Rightarrow E$. The mean stress becomes zero (do not get confused: the primary stress is not zero, because the primary stress is still applied. The stress cycle changes to that with no mean stress.).

Now, we look at this in terms of a combination of the stress amplitude and the mean stress. If as a result of elastic analysis, point A is obtained as shown in Fig. 5.20,

or in the case of Fig. 5.19a, the elastic-perfectly plastic material does not yield, and therefore the mean stress and the stress amplitude are still at point A. If point B is obtained, or in the case of Fig. 5.19b, the mean stress shifts to point B*. If point C is obtained, it shifts to point C*. This can be described by the following equations:

- σ_{mean} : mean stress directly obtained from the load cycle by elastic analysis
- σ_{mean}^* : adjusted mean stress (the mean stress in the elastic-perfectly plastic material from the second cycle onward)
- σ_{alt} : amplitude in alternating stress cycles
- S_y : yield strength (or yield stress, yield point)

- (1) If $\sigma_{\text{mean}} + \sigma_{\text{alt}} \leq S_y$, $\sigma_{\text{mean}}^* = \sigma_{\text{mean}}$ (for example, point A in Fig. 5.20)
- (2) If $\sigma_{\text{mean}} + \sigma_{\text{alt}} > S_y$ and $\sigma_{\text{alt}} < S_y$, $\sigma_{\text{mean}}^* = S_y - \sigma_{\text{alt}}$ (for example, point B* in Fig. 5.20)
- (3) If $\sigma_{\text{alt}} \geq S_y$, $\sigma_{\text{mean}}^* = 0$ (for example, point C* in Fig. 5.20).

The number of cycles to fatigue life should be calculated for the adjusted mean stress to which it shifts from the second cycle onward. However, a design fatigue curve is obtained as the relationship between the stress amplitude and the life cycle from alternating strain- or load-controlled fatigue test data with zero mean stress. We describe the reduction in fatigue life due to the mean stress by increasing the stress amplitude.

First, it can be conservatively described by the failure line (or fatigue fracture line) in Fig. 5.21 how the endurance limit stress amplitude changes due to the mean stress in high-cycle fatigue regime. Under fully alternating fatigue test, $\sigma_{\text{mean}} = 0$ and for endurance limit, $\sigma_{\text{alt}} = \sigma_e$. For $\sigma_{\text{mean}} = S_u$ (tensile strength), $\sigma_{\text{alt}} = 0$. To be conservative, a straight line is drawn between these two points. Fatigue failure does not occur under fatigue test conditions below this line in the figure.

In low-cycle fatigue regime, when yielding occurs, the mean stress shifts and then the adjusted mean stress decreases to zero, as shown in Fig. 5.19c. As a result, the fatigue life does not change.

Which state with zero mean stress is equivalent to fatigue damage due to this stress condition can be determined by the method in Fig. 5.21. If point A is obtained for the mean stress by elastic analysis, yielding occurs and therefore the mean stress shifts to point A*. A line through point A* is drawn from the intercept S_u of the horizontal axis. The value of σ_{eq} is the intercept of this line with the vertical axis, and is given by.

$$\sigma_{\text{eq}} = \frac{\sigma_{\text{alt}}}{1 - \frac{\sigma_{\text{mean}}^*}{S_u}} \quad (5.42)$$

Elastic analysis shows that the stress amplitude (the equivalent stress amplitude) σ_{eq} with zero mean stress causing the same fatigue damage as due to stress cycles with σ_{mean} and σ_{alt} can be conservatively calculated by the above equation and is naturally higher than σ_{alt} . This is called the modified Goodman's diagram method.

In structures in service, residual stress is present in welds and affects fatigue life in terms of the mean stress. However, the residual stress is difficult to identify. In the current structural codes, the mean stress that decreases fatigue life most is calculated and the allowable stress amplitude component is set as the design limit. The maximum mean stress and the corresponding allowable stress amplitude are calculated as shown in Fig. 5.22.

In an elastic-perfectly plastic material, the combined value of the adjusted mean stress σ_{mean}^* (the value after the shift) after yield and the stress amplitude σ_{alt} is in the triangle OACB and above point A on the vertical axis (that is, $\sigma_{\text{alt}} \geq S_y$, corresponding to Fig. 5.19c). Fatigue fracture does not occur below the failure line as given in Fig. 5.22. Therefore, the range of the combination of σ_{mean}^* and σ_{alt} that can cause fatigue fracture is only the shaded triangle area denoted as “fatigue failure region” and above point A on the vertical axis. This means that the maximum mean stress likely to cause fatigue fracture is the value at point C. What is the stress amplitude for the mean stress shown as point C in Fig. 5.22 when the stress amplitude with no mean stress to cause fatigue fracture is S_N ? The modified Goodman diagram tells us that when the stress cycles with stress amplitude σ_{alt} and mean stress σ_{mean} are applied, the number of cycles to cause fatigue fracture is conservatively determined as that for the stress cycles of stress amplitude σ_{eq} on the fatigue curve that is obtained from fatigue tests with no mean stress as shown in Fig. 5.22. As mentioned above, the failure line (or fatigue fracture line) describes how the endurance limit stress amplitude changes due to the mean stress in high-cycle fatigue regime. Accordingly, when the allowable stress amplitude is S_N for no mean stress, the allowable stress amplitude for the mean stress shown as point C becomes $S_{N'}$ in Fig. 5.22, and is given as Eq. (5.43).

For $S_N < S_y$

$$S_{N'} = S_N \left[\frac{S_u - S_y}{S_u - S_N} \right] \quad (5.43)$$

A mean stress is induced in weldment and other structural discontinuities, and therefore, a fatigue life reduction due to the mean stress should be taken into consideration. On the basis of the above discussions, design of the weldment and discontinuities is recommended to consider the fatigue life reduction to the value at point C, namely, $S_{N'}$, as the potential maximum reduction.

In the current structural codes, design fatigue curves are obtained from best-fit curves modified taking into account the maximum effect of mean stress.

(3) Stress cycle counting method applied to an irregular stress history experienced in a real component (many types of stress cycle with different stress amplitudes)

A time history of stress and strain experienced in a real component (called a stress/strain history, stress/strain wave form) that is specified for fatigue analysis is irregular, differing from stress/strain histories with constant stress/strain amplitude in fatigue tests. In particular, in low-cycle fatigue regime, such as thermal fatigue produced by

repetitions of thermal transient events (for example, startup, shutdown and reactor trip events), the irregularity is pronounced, because the stress/strain transient response is different for each event. However, it is not practical to conduct fatigue tests of a real structural element with such irregular stress/strain histories. Thus, a great deal of effort have been devoted in developing an evaluation procedure of fatigue damage to be experienced in a structural element subjected to a real stress/strain history by using results of fatigue tests with constant total strain ranges or constant ranges of other parameters.

Here, the rainflow counting method is introduced as frequently utilized cycle counting method in the industry and was proposed by Japanese researchers.

In the following, first, fatigue tests, which are introduced in Chap. 3, are mentioned with focusing on key points for introducing the rainflow method. Particularly, multi-stage fatigue tests are mentioned, because of their importance for investigating a combination effect of different stress/strain cycle patterns. Next, key points of the rainflow method are described.

(i) Fatigue tests

In the low-cycle fatigue regime, usually up to 10^4 cycles, fatigue tests are conducted with a constant total strain range ($\Delta\epsilon_t$), strain-controlled and fully alternating wave form. The strain waveforms given as a function of time include of a triangular type and a sinusoidal type as shown in Fig. 5.23a-1. As mentioned above, a best-fit curve is obtained from these fatigue test data, as representing a number of cycles to cause fatigue fracture as abscissa axis and the applied total strain amplitude or apparent stress amplitude as ordinate axis. The total strain amplitude ($\Delta\epsilon_t$)/2 can be converted to the apparent stress amplitude (as elastically calculated stress) S or (ΔS)/2 by the following relationship: $\Delta S/2 = E \times \Delta\epsilon_t/2$. By using the technical terms used in design codes, the curve is represented by a correlation of the number of allowable cycles and the alternating peak stress intensity S_p or simply, the alternating stress intensity S_{alt} . In Fig. 5.23, these terms are used for easily imaging the design evaluation procedure. Under a constant total strain range, the corresponding stress range varies with the number of applied cycles. Figure 5.23a-2 schematically illustrates how the stress ranges vary in a cyclic-hardening material. A stress-strain curve at the individual cycles, called the “hysteresis loop”, is depicted in Fig. 5.23a-3. Many austenitic stainless steels are cyclically hardened materials, while we can observe cyclically softened in many carbon steels and low-alloy steels.

It is required basically in fatigue damage evaluation that the number of strain cycles with the above-mentioned wave form be below the average number of cycles to cause fatigue fracture N_F as shown in this figure. It should be noted here that in this figure, this average number of cycles as the allowable number of cycles, in order to easily image a concept of design fatigue curve. In design codes, the allowable number of cycles is obtained from the design fatigue curve that is generated from the best-fit curve or the average number of cycles to cause fatigue fracture by applying the design factors of 20&2 and modification by the mean stress effect. The number of cycles to cause fatigue fracture is defined the number of cycles at which the stress amplitude applied to the fatigue specimen decreases by 25% from that at the maximum and

saturated stress cycling region, where the stress amplitude remains unchanged with cycling. At this 25% stress drop, a fatigue crack propagates in the cross section of specimen. The definition of fatigue fracture by the 25% stress drop may be called 25% descent method, as mentioned in equation (5.41).

Next, a basic concept of fatigue damage evaluation for multi-step fatigue tests, in particular, the two-step fatigue tests, as given in Figure 5.23b. The maximum number of cycles with the individual stress amplitudes to be applicable to the fatigue test, n_1 and n_2 , respectively, can be determined using the Miner rule, well known as fatigue damage accumulation evaluation rule, as shown in this figure. Similarly, fatigue damage of multi-step strain cycling can be evaluated using the Miner rule from the fatigue curve generated from constant strain range fatigue test data.

(ii) Rainflow method

As mentioned above, waveforms of stress and strain experienced in a real structural component are significantly different from those of multi-step fatigue tests. A typical example is shown in Chap. 7: seismic acceleration waveforms. Figure 5.24a conceptually illustrates several patterns of strain–time waveforms that may be observed at events of normal startup, shutdown and abnormal reactor trip/scram. This figure involves exaggeration of the degree of plasticity, in order to make it easier to understand the rainflow method. Practically, at normal startup and shutdown, a plasticity may not be developed, because of elastic design for service conditions A and B. Repetition of the same strain range cycle in a row, similarly to alternating strain cycle pattern of fatigue tests, rarely happens in a real component. An exception is repetition of rated power operation with no trouble.

In order to evaluate fatigue damage of a real component, therefore, it is required to count strain ranges and their cycles to give equivalent representation of fatigue damage from a real strain–time history. One of frequently used cycle counting methods is the Rainflow counting method developed by Endo et al. [15]. The procedure of this method is given as one of standard practices for cycle counting in ASTM standards E 1049-85 [16]. The rainflow method was originally proposed, which is called “rainflow method in a narrow sense,” aiming at determining strain amplitudes and their number of cycles which contribute initiation of fatigue crack, and focuses on formation of hysteresis loop. This method provides identification of peaks and valleys (or troughs) and number of cycles contributing the initiation of fatigue crack for irregularly variable stress/strain–time histories.

In detail, stress/strain amplitudes and number of their cycles are determined as follows:

- (a) First, the time axis of strain–time history (or strain waveform) as shown in Fig. 5.24a is set downward as vertical axis (to facilitate understanding of this method from an analogy of rain falling down a pagoda roof). An inclinatory portion of strain waveform, representing the portion where starting at each peak (or valley (or trough)), strains continuously vary with time toward the next valley (or peak) is regarded as a roof. Definitions of peak and valley (or trough) are as follows:

Peak; the derivative of the stress/strain-time history changes from positive to negative sign.

Valley (or trough); the derivative of stress/strain-time history changes from negative to positive sign.

A rainflow is started at each base (the highest point of each roof). The base is called “water source.” Water source is valley or peak. The rain flowing down a roof, and comes to a tip of the roof, falling on the next roof. Spatial relations of roofs determine forms of hysteresis loop, that is, locations of the water source of a rainflow and the tip of rain flowing roof, and the spatial relations of roofs over the roof under consideration and those under roofs determine which part of a hysteresis loop the rainflow forms and whether or not the roof tip point becomes the vertex of the hysteresis loop. Note that in the following discussion, regimes of constant stress/strain in stress/strain-time history are deleted because of no influence on fatigue damage.

- (b) Rains flowing down a roof to the right are discussed here. Rainflow to the left may be discussed conversely.
 - (I) Condition of a roof tip point being a vertex of hysteresis loop when a rain falling from the roof tip, and therefore, the rain falling down stops: consider a rain flows on a roof, starting from a water source (hereinafter called starting roof and water source). If a water source on a roof under the starting roof (for example, next to the starting roof) is located on the left side of starting water source, stop the rain flowing and falling at the tip of the roof which is one just over this roof (as shown as 1.5 in Fig. 5.24b). In other words, when a rainflow path that is started at the starting water source comes to a tip of the roof, the flow stops if the opposite water source is more negative than the one the flow started from. Then, the tip of the roof where the rainflow and rainfall stop becomes the vertex of hysteresis loop. If a water source is located on the right side of the starting water source or at the identical stress/strain to that of the starting water source, continue the rainflow. This may be counted as a half cycle.
 - (II) Prevention of flow mixing of rainflows: If a rain fall from a roof over the roof under consideration is mixed with the rainflow from the water source on the roof under consideration, or a rain from a roof over the roof under consideration falls just to the tip of the roof under consideration, the rainflows on the roof under consideration is stopped at the mix point, and then the rain from the roof over the roof under consideration continues flowing. In other words, if the rain flowing down the roof under consideration intercepts flow from an earlier path, the present path (or the rainflow from the water source on the roof under consideration) is stopped. In a spatial relation of roof and water source to satisfy the above condition (I), the rainflow on the roof under consideration continues after the mix point, while the rain fall from the roof over the roof under consideration is stopped. In a spatial relation to meet the condition (II), a rainflowing to the left meets the condition (I), and therefore, this may be counted as a full cycle because

a full cycle of hysteresis loop is formed (or the hysteresis loop is closed) ranging from the tip of the roof over the roof under consideration to the water source on the roof under consideration.

Figure 5.24b illustrates how to count a cycle and to form a hysteresis loop. A good applicability of the rainflow method to a complicated and irregular stress/strain history to be experienced in a real component is clearly shown.

In addition, as described in [4] in Sect. 5.6.2, an evaluation method for $P + Q \geq 3S_m$ called the simplified elastic-plastic analysis method using the factor K_e is provided in several structural codes and standards.

5.6.4 Buckling Limits

In this subsection, describe a concept of method for preventing buckling of a structural component subjected to external pressure using external pressure buckling charts, so-called factors A and B charts. Rules of this method that are usually prescribed in design codes or in allowable stress sections in material codes. They are provided in not only the codes on the basis of the Design by Analysis approach but also those of the Design by Formula approach, because the buckling should be explicitly prevented in both approaches.

The pressure (critical pressure) p_{cr} that causes the buckling of an infinitely long and thin-walled cylinder subjected to an external pressure is discussed as the simplest case. The critical pressure of the cylinder under elastic buckling conditions is expressed as follows:

$$p_{cr} = \frac{2}{(1 - \nu^2)} E \left(\frac{t}{D} \right)^2 \quad (5.44)$$

where E : Young's modulus, ν : Poisson's ratio, t : minimum thickness of the shell, D : diameter of the shell.

Similarly to Eq. (5.2), the circumferential stress σ_{cr} (in an absolute value) is given by

$$\sigma_{cr} = \frac{D}{2t} p_{cr} \quad (5.45)$$

Then,

$$\sigma_{cr} = \frac{E}{(1 - \nu^2)} \left(\frac{t}{D} \right) \quad (5.46)$$

The allowable pressure p_a is given by the following equation in terms of the design factor DF:

$$p_a = \frac{p_{cr}}{DF} = \frac{1}{DF} \frac{2t}{D} \sigma_{cr} \quad (5.47)$$

Values of the design factors DFs are given in Item [3] in Sect. 5.3.2. When the factors A and B are defined as follows,

$$A = \frac{\sigma_{\theta,cr}}{E} : \text{circumferential elastic strain due to critical pressure} \quad (5.48)$$

$$B = \frac{\sigma_{\theta,cr}}{2} \quad (5.49)$$

Allowable pressure p_a is obtained from the following expression in terms of the factor B,

$$p_a = \frac{4}{DF} B \left(\frac{t}{D} \right) \quad (5.50)$$

Now, we discuss an elastic buckling, and then the factors A and B have the following proportional relationship,

$$B = \frac{1}{2} EA \quad (5.51)$$

In a case of finite cylindrical shell, A is given as follows,

$$A = \frac{\sigma_{\theta,cr}}{E} = \frac{1.30 \left(\frac{t}{D_0} \right)^{3/2}}{\frac{\ell}{D_0} - 0.45 \left(\frac{t}{D_0} \right)^{1/2}} \cong \frac{1.30 \left(\frac{t}{D_0} \right)^{3/2}}{\frac{\ell}{D_0}} \quad (5.52)$$

where ℓ is the length of the shell and D_0 the outer diameter [17].

The above discussion is about the elastic buckling of cylindrical shell subjected to external pressure.

Next, in a case of elastic–plastic buckling, a simplified method for estimating the critical pressure or critical circumferential stress $\sigma_{\theta,cr}$ is to use a tangent modulus E_r obtained from a stress–strain curve for the material under consideration in place of a modulus of elasticity. In the ASME Boiler and Pressure Vessel Code Section II Part D Subpart 3 [18], $\sigma_{\theta,cr}$ is given by the following expressions,

$$\begin{aligned} \sigma_{\theta,cr} &= \frac{E_r}{E} \alpha \sigma_e \\ A &= \frac{\sigma_{\theta,cr}}{E_r} = \frac{\alpha \sigma_e}{E} = f \left(\frac{t}{D_0}, \frac{\ell}{D_0} \right) \\ B &= \frac{\sigma_{\theta,cr}}{2} \end{aligned} \quad (5.53)$$

where α is the knockdown coefficient describing the difference between the theoretical and experimental values of buckling stress, due to imperfections (ovality, etc.) and others in structural configuration not considered in a theoretical solution (Eq. (5.44), etc.).

The critical pressure p_{cr} can be derived in terms of $\sigma_{\theta,cr}$ above. The factor A is determined as a function of t/D_0 and ℓ/D_0 at the individual temperature. The function is obtained from results of buckling experiments.

As clearly seen from the above expressions, the factor A represents the term that is determined as the function of the configuration of the structural component but regardless of the material. By multiplying the factor A by the tangent modulus E_t , the circumferential stress at the critical pressure $\sigma_{\theta,cr}$ is obtained as the factor B.

In the current structural codes, evaluation of buckling under external pressure uses external pressure charts represented by the factors A and B as discussed above. In the ASME Boiler and Pressure Vessel Code, external pressure charts are provided in Section II, Part D, Subpart 3 [18], which provides material specifications and allowable stresses applicable to both nuclear and fossil-fuel power plant components. The JSME Rules on Design and Construction for Nuclear Power Plant are the same in this regard.

5.7 Rules Specific to Components (Stress Index Method for Class 1 Piping)

This section describes a piping design method using the concept of stress index.

5.7.1 Stress Index Method

The Design by Analysis approach discussed above is applicable to general operating conditions and configurations of a structural component. A detailed stress analysis is carried out for given structural configurations and applied loads, and using the stress analysis result, a structural evaluation including sizing is performed.

A pressure vessel is a kind of custom-ordered and built to order component, and therefore, its inner diameter and other dimensions are not standardized. In addition, loading conditions are complicated. Hence, it is adequate to apply the above-mentioned Design by Analysis approach to a design of vessel.

However, the total length of piping systems in a plant reaches several kilometers or above, and they have a wide variety of internal pressure conditions and piping diameters. Further, a piping system consists of several types, for example, straight pipes, elbows and tees. Straight pipes between piping components have different lengths. It is a hard task to apply the above-mentioned generalized Design by Analysis approach to a design of piping systems. On the other hand, kinds of loading acting

on the pipes to cause a significant damage may be limited, comparing to those of vessels. Further, types of piping are relatively limited and therefore, it may be possible to simplify a stress evaluation procedure. Accordingly, it may possible to classify diameters and wall thicknesses of pipes step by step in accordance with pressure and temperature (precisely speaking, the design pressure and temperature). We call such a classification “pressure–temperature rating.” Sizing and structural integrity evaluation can be done for limited kinds of loading for the representative wall diameter and thickness of each classification, by directly inputting values of loads to be evaluated (for example, internal pressure, bending moment and thermal loads) into relevant evaluation formulae. Taking into account the characteristics specific to piping (typically, limited types of loading and also limited types of piping), integrity evaluation formulae can be developed by applying so-called stress index concept.

Even the stress index method, the structural integrity is basically evaluated using Eq. (5.1). However, the stress to be evaluated (L in Eq. (5.1)) is calculated automatically by multiplying representative stress due to applied loads by the relevant “stress indices” that are specified for limited types of piping. This method is conceptually expressed as follows:

$$\begin{aligned} \text{Stress to be evaluated} &= \text{Representative stress due to individual load} \times \text{Stress index} \\ &\leq \text{Allowable stress} \end{aligned} \quad (5.54)$$

For example, S_p used in fatigue evaluation is

$$\begin{aligned} S_p &= \text{pressure term} + \text{moment term} + \text{thermal load terms} \\ &= K_1 C_1 (PD_0/2t) + K_2 C_2 M_i/Z + K_3 E \alpha |\Delta T_1|/(2(1 - \nu)) \\ &\quad + K_3 C_3 E_{ab} |\alpha_a T_a - \alpha_b T_b| + E \alpha |\Delta T_2|/(1 - \nu) \end{aligned} \quad (5.55)$$

These stress indices are defined as follows:

- Primary-plus-secondary stress index: C_1 for pressure, C_2 for a moment, C_3 and C_3' for a temperature gradient
- Peak stress index: K_1 for pressure, K_2 for a moment, K_3 for a temperature gradient.

The total stress index is $K_i C_i$.

These factors were determined from the results of extensive research and development.

It should be mentioned here that for the vessel design by applying the Design by Analysis approach, first, stresses are calculated from applied loads on the assumption of elastic material, and then the calculated stresses are compared to the relevant allowable stress limits.

The following features of the stress index method may be summarized:

- Stress limits have been developed to simply apply the stress index concept to elbows, tees and other fittings as well as straight pipes, by calculating the stress in a straight pipe of the same diameter and wall thickness as the fitting and multiplying

the stress value by a stress index to calculate the stress in the fitting (as mentioned above, the stress in a straight pipe is calculated by inputting the load). The stress indices vary depending on the relevant failure mode.

- Bending is the primary concern in a piping system design. A minimum wall thickness requirement and general primary membrane plus bending stress limit play an important role as primary stress limits. It should be noted here that for safety design, a double ended guillotine break (DEGB) is postulated as a pipe break at the event of LOCA. For DEGB, it is postulated to break the pipe into two pipe pieces by tearing the pipe in the circumferential direction (perpendicular to the axial direction). This pipe break may be caused by bending stress in the axial direction. A break or crack propagation is caused in the direction perpendicular to the direction of governing stress component. On the other hand, a vessel is torn in the axial direction, because the circumferential stress component induced by internal pressure is governing or highest.

5.7.2 Stress Limits for Class 1 Piping (Representative Stress Limit Equations [1, 2])

A major part of the class 1 piping design provided in Ref. [2], NB-3650 “analysis of piping products,” and the corresponding part in Ref. [1], PPB-3500 “design by analysis for piping system” are described below. In these piping design rules, the design related to the allowable pressure limit is referred to Ref. [2], and symbols of stress limit expressions are referred to both Refs. [1, 2].

- (1) Pressure design of straight pipe under internal pressure (or the calculated wall thickness t)

The calculated wall thickness (or pressure design thickness) t of a straight pipe subjected to internal pressure shall be determined as follows,

$$t = \frac{P \times D_0}{2(S_m + yP)} \quad (5.56)$$

where P is the internal Design Pressure, S_m the maximum allowable stress intensity for the material at the Design Temperature, and y the coefficient which varies with temperature; $y = 0.4$ for 480 °C or below. The minimum required thickness of a pipe wall, t_m , is specified in Ref. [2], NB-3641, and $t_m = t + A$, where A is the additive thickness including corrosion or erosion allowance.

- (2) Design conditions and Levels A and B service limits

- (i) Primary stress intensity limit for design conditions

$$S_{pm} = B_1 \frac{PD_0}{2t} + B_2 \frac{M_{ip}}{Z_i} \leq 1.5S_m \quad (5.57)$$

where M_{ip} is the resultant moment due to a combination of Design Mechanical Loads. Z_i is the section modulus, expressed as $Z = I / (D_0 / 2)$, where I is the modulus of inertia and D_0 the outside diameter of pipe.

Primary stress index: B_1 for pressure, B_2 for a moment.

- (ii) Primary plus secondary stress intensity range limit for Levels A and B services

$$S_a = C_1 \frac{P_0 D_0}{2t} + C_2 \frac{M_{iS}}{Z_i} + C_3 E_{ab} |\alpha_a T_a - \alpha_b T_b| \leq 3S_m \quad (5.58)$$

If the above equation is not met, the piping product may still be satisfactory, provided that the condition of NB-3653.6 in Ref. [2], Design by Analysis. The definitions of E_{ab} , T_a , T_b and others are omitted here.

- (iii) Peak stress intensity limit for Levels A and B services

The peak stress intensity is derived from the highest value at any point across the thickness of a section of the combination of all primary, secondary, and peak stresses (Ref. [2], NB-3222.4(b)). For a type of stress cycle, denoted as I, in cyclic application of loads and thermal conditions, calculate the usage factor U_i as the ratio of n_i / N_i , where n_i is the specified number of times that the type I of stress cycle will be repeated during the life of the component, and N_i the maximum number of repetitions which would be allowable, determined from the alternating peak stress intensity S_i using the applicable design fatigue curve.

Next, calculate the cumulative usage factor from $U = \sum U_i = \sum n_i / N_i$, and the cumulative usage factor U shall not exceed 1.0. In other words,

$$\sum \frac{n_i}{N_i} \leq 1.0 \quad (5.59)$$

The alternating stress intensity S_i is equal to one-half the value of the peak stress intensity (as the total stress) S_p as expressed below

$$S_p = K_1 C_1 \frac{P_0 D_0}{2t} + K_2 C_2 \frac{M_{iS}}{Z_i} + K_3 \frac{E\alpha |\Delta T_1|}{2(1-\nu)} + K_3 C_3 E_{ab} |\alpha_a T_a - \alpha_b T_b| + \frac{E\alpha |\Delta T_2|}{1-\nu} \quad (5.60)$$

(3) Level C service limits

- (i) Permissible pressure

The calculated maximum allowable internal pressure acting on the inside surface of pipe or permissible pressure shall not exceed the internal Design Pressure (or the pressure calculated in accordance with Eq. (5.56) for the straight pipe) by more than 50%.

(ii) Primary stress limit

The condition of Eq. (5.57) using the allowable stress $2.25S_m$ but not greater than $1.8S_y$ in place of $1.5S_m$ shall be met. In other words,

$$S_{pm} = B_1 \frac{PD_0}{2t} + B_2 \frac{M_{ip}}{Z_i} \leq \min.(2.25S_m, 1.8S_y)$$

$$2.25 = 1.5 \times 1.5 \quad (5.61)$$

(4) Level D service limits

(i) Permissible pressure

The permissible pressure acting on the inside surface of pipe or shall not exceed 2.0 times the Design Pressure (or the pressure calculated in accordance with Eq. (5.56)).

(ii) Primary stress limit

The condition of Eq. (5.57) using the allowable stress $3S_m$ but not greater than $2S_y$ in place of $1.5S_m$ shall be met. In other words,

$$S_{pm} = B_1 \frac{PD_0}{2t} + B_2 \frac{M_{ip}}{Z_i} \leq \min.(3S_m, 2S_y) \quad (5.62)$$

In the above equation, the representative stress due to internal pressure is $PD_0/2t$ and appears to represent the circumferential stress in a cylindrical pipe. Since $B_1 = 0.5$, however, the representative stress as the pressure term represents the axial stress due to internal pressure.

5.8 Design by Analysis Using the Finite Element Method

Today, analytical techniques using the finite element method (FEM) described in Chap. 4 are mature, and finite element analysis (FEA) is commonly employed. However, this is a recent trend. Until then, the strength of materials formulas described in Chap. 2 had been used for hand calculation of stress, strain, and deformation. The Design by Analysis approach described in Sect. 5.6 requires a detailed stress analysis. In the 1960s where the Design by Analysis approach was incorporated into codes, the available stress analysis methods were the discontinuity analysis method and the equilibrium analysis method. The Design by Analysis approach was developed for use in these analysis methods. In the discontinuity analysis method, a structure is divided into elements of appropriate size and geometry and the stress and others in each element are calculated. The external forces and moments on the element are determined so that the discontinuity of relative displacement and rotation between the elements does not occur, or the relative displacement and rotation between elements are structurally compatible. With these external forces and moments on the ends of the element, the stress and others in the element are calculated

by the strength of materials formulas in Chap. 2. Since strength of materials formulas are used, a stress classification, such as primary stress, secondary stress, and peak stress, membrane stress, and bending stress, was straightforward. However, hand calculation using strength of materials formulas had limitations when the formulas were applied to structures of complicated configurations and loadings with a complex distribution, requiring an effort to allow them to be used for approximate calculation.

The FEM requires no such effort, making it easy to evaluate stress even in a structure with complicated configuration and subjected to complex loading. When shell elements are used, membrane and bending stress components can be obtained as output data. However, when solid elements, most suitable for complicated three-dimensional configurations, are used, output data are not broken down into membrane and bending stress components, particularly for structural discontinuities. It needs efforts to classify the outputted stress into primary membrane plus bending stress, secondary stress and peak stress. Because of this, it was desirable to develop a design method suitable for FEA-based design. This issue was strongly recognized in the 1980s when FEM analysis techniques became mature.

The following two major FEA-based design methods were proposed:

- (a) A method to evaluate FEA stress analysis results according to the requirements of the codes

It is necessary to develop guidelines for how to use FEA to evaluate stress for each failure mode and how to classify stresses.

- (b) A method to directly evaluate FEA results without classifying stresses while recognizing the relationship between the failure modes required by the codes and stress categories

The method in (a) was extensively discussed mainly in the United States. In the United States, the Pressure Vessel Research Council (PVRC) systematically studied it and published guidelines in 1998 [19]. The method in (b) was discussed in Japan, the United States, and Europe. In Japan, it is included in a code case in the JSME Rules on Design and Construction for Nuclear Power Plant [20]. The key points of these methods are described below.

5.8.1 When the Stress Classification in the Current Codes is Used

The following must be performed to ensure that the stresses from FEA meet the stress limits in the current codes [19].

- (i) It must be clearly understood which failure mode the stress limit is intended to prevent.
- (ii) A method for classifying the stress distribution from FEA into primary and secondary stresses must be developed.

FEM analysis using solid elements is useful to calculate the primary bending stress intensity P_b , the local primary membrane stress intensity P_L , and the primary plus secondary stress intensity range $(P + Q)$ for a structure of complicated configuration. However, it is not necessarily suitable for calculation of the general primary membrane stress intensity P_m . It is suitable to calculate P_m using formulas provided in the codes and equilibrium analysis.

- (iii) A method for defining the location to be evaluated for the purpose of classification into stress categories must be developed.

Reference [19] defines the line and plane to be evaluated for two- and three-dimensional structures as a stress classification line (SCL) and a stress classification plane (SCP), respectively, and provides guidelines for identifying them.

- (iv) A method for calculating membrane and bending stress components must be developed.

The PVRC investigated these methods and proposed guidelines, which are briefly described below.

[1] The relationship between the failure modes and the stress categories

This is described in the previous sections, but is briefly described here again.

- General primary membrane stress intensity P_m : the main failure modes due to this stress intensity are plastic tensile instability, and ductile fracture.
- Primary bending stress intensity P_b : the failure modes due to P_m are caused by P_b , too, except plastic tensile instability. A plastic collapse caused by forming a plastic hinge is considered as a mode of failure. The biggest difference with P_m is that through-thickness yielding requires a higher stress compared to P_m , because of $M_c > M_y$. This is because P_m is a membrane or uniformly distributed stress and the stress distribution is described by a point stress (namely, uniform at the point stress across the thickness) while P_b has such a different feature that when a bending moment causes yielding on the surface, the interior is still in an elastic region. Hence, it is possible to further increase the bending moment up to that to cause the through-thickness yielding, namely, M_c . The effect of a two-dimensional area must be considered. This is called the shape-factor effect as described in Item [2] (3), in Sect. 5.6.1.
- Primary local membrane plus bending stress intensity $(P_L + P_b)$: the main failure mode is excessive plastic deformation.
- Primary plus secondary stress intensity range $(P + Q)$: the $(P + Q)$ limit ensures that the membrane plus bending stress changes within the elastic range or that the material shakes down to an elastic cycle. This limit is intended to warrant an fatigue analysis based on elastic stress calculation.

[2] Stress calculation by category

- (i) P_m : Code-provided equations should be used. It is desirable to avoid, for economic reasons, to obtain P_m from FEA. For complex structures to which

code-provided equations cannot be applied, it is desirable to use limit analysis or plastic analysis, instead of linear elastic analysis.

- (ii) P_L , P_b or Q : Care should be taken when using FEA to evaluate primary stress components at structural discontinuities because secondary stress could be evaluated mistakenly as primary stress. Equilibrium analysis and plastic analysis are desirable.

[3] Evaluation location

Structural elements are divided into basic and transition elements. The basic element is an element forming the basic unit of a structure. The transition element is an element connecting the basic elements. The membrane plus bending stress is evaluated for the basic element. Fatigue evaluation is done for the transition element. There are two cases to consider:

- (a) When the mesh size is not sufficiently small to calculate the total stress in fatigue evaluation

The total stress is obtained from the membrane plus bending stress using the stress concentration factor (SCF).

- (b) When the limit on $(P + Q)$ is not met in the transition element

The factor K_e (also called the penalty factor) in Eq. (5.36) is applied.

[4] Calculating membrane and bending stresses

Membrane and bending stresses are obtained from stress components as follows, instead of principal stress:

- (i) An internal force and a moment are obtained by integrating the distribution of each stress component at the evaluation location (a line (SCL) or a plane (SCP)).
- (ii) Membrane and bending stress components are obtained as P/A and $6M/t^2$ from the internal force P and the moment M .
- (iii) For the membrane and bending stress components obtained for each stress component σ_{ij} , the principal stresses are obtained. The stress intensity range is determined by obtaining the range of each stress component σ_{ij} . The principal stress range is not required. The purpose of obtaining the range of each stress component σ_{ij} is to avoid the need to check the rotation of the principal stresses.

5.8.2 Direct Evaluation by FEA

It is not easy to obtain a stress distribution in a structure of complex configuration or a structure under complex loading using FEA and classify stress into the stress categories provided in the codes. The reason for classifying stress is probably that failure prevention can be linked to the tensile strength, the yield stress, and the number

of cycles to fatigue failure and that it is appropriate to describe the basic principle [Eq. (5.1)] of integrity evaluation in terms of stress. There is also an approach in which failure of pressure-retaining components is evaluated in terms of pressure or external loading, instead of stress.

As an alternative approach applicable when stress is difficult to classify, an approach was investigated in which evaluation is performed in terms of external loading, particularly monotonic loading. In one typical such approach, the limit load or the plastic collapse load is calculated as the load to cause failure (called failure load), which is used as the allowable value to limit the load applied. The allowable load is derived by dividing the failure load by the specified design factor, as easily imaged by Eq. (5.1). The limit load and the plastic collapse load are described below. These loads correspond to the category of primary stress. The limit on monotonic loading may be called the primary load limit.

[1] Calculating the limit load

A structure under monotonic loading has a non-uniform stress distribution even if the material is elastic-perfectly plastic. Therefore, yielding occurs first at the point with the highest stress. This is point A in the load–deflection curve in Fig. 5.25. With increasing load, the yielded region extends to a substantial size. However, the material is still constrained by the surrounding elastic region, preventing significant plastic deflection as the structural behavior. With a further increase in load, the yielded region further extends while the surrounding elastic region shrinks to a size insufficient to constrain the yielded region. Then, the gross plastic deformation of the structure occurs. The limit load is defined as the load at which gross plastic deformation occurs [21]. For elastic-perfectly plastic or rigid-plastic materials, the limit load is defined as the load on the asymptotic line as shown in Fig. 5.25. Therefore, the load cannot be further increased beyond this limit.

In summary, the limit load is the maximum load that can be applied to a structure under the following conditions:

- (a) The small displacement theory is applied to a displacement–strain relation. Then, strain is given by a first-order differential equation for displacement.
- (b) The material response is assumed to show a rigid-plastic or elastic-perfectly plastic response, with an admissible yield function.
- (c) The internal stresses and the applied forces meet the usual equation of equilibrium which ignores changes in geometry due to deformations.

The limit load is significant when the material is sufficiently ductile and its sensitivity to small notches can be ignored. It is insignificant when geometrical softening occurs as a result of increasing deformation and then plastic instability occurs. In this case, the plastic instability load should be obtained by a large displacement elastic–plastic analysis.

As described in Item [2] (2) in Sect. 5.6.1, in the system of allowable stress based on stress classification, the allowable range is wider for bending stress than membrane stress due to the effect of shape factor. The analysis required is a limit

analysis. The shape factor represents the ratio of the limit load. Many criteria in the structural codes and standards are based on the limit load from limit analysis.

[2] Plastic collapse load

While the limit load is obtained from small displacement analysis of an elastic-perfectly plastic material, the plastic collapse load is obtained by means of plastic analysis. In calculating it, the effect of the configuration change of a work-hardening material due to large displacement can be taken into account.

The plastic collapse load is thought to be a generalized concept of a plastic hinge that is produced in a structure subjected to a bending moment on the assumption of an elastic-perfectly plastic material. It is the load that causes significant plastic deformation of a structure as a whole and is intended to be the load at which the surrounding elastic region can no longer stop the progress of gross plastic deformation due to the extension of the plastic region. Therefore, while the yield strength is the only material property required for analysis to obtain the limit load, the work-hardening coefficient as well as the yield strength, is required to obtain the plastic collapse load.

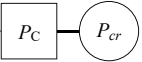
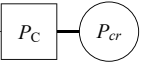
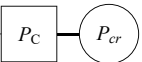
The term “plastic collapse load” may be misleading in the context of what the term “collapse” means. Because of this, it was proposed to replace it with another term such as “plastic load” [19].

The plastic collapse load can be obtained by the twice-elastic-slope (or double-elastic-slope) method. The twice-elastic-slope method is the method to obtain the plastic collapse load as the intersection between the load–displacement curve and the line OP with a gradient ($\tan\phi = 2 \tan\theta$) twice the gradient ($\tan\theta$) of line OA in the elastic region, as shown in Fig. 5.26.

As you can see in Fig. 5.26, the plastic collapse load obtained by this method is not directly related to fracture or failure. This holds true for the limit load. In order to directly relate to fracture or failure, the effect of the design tensile strength S_u is considered by defining the yield stress σ_y , which represents the stress at the bending point of a stress–strain curve, in terms of S_m , for example, as $\sigma_y = 1.5S_m$. For service conditions A and B where elastic design is applied, this operation is effective. For service condition D where some plastic deformation is allowed, the operation is more complicated. The method in the current codes uses a stress value at the bending point that takes into account such an effect (Table 5.11). However, it is better to obtain the load at which fracture or failure occurs and apply a design factor to it.

The method discussed above is a method for determining primary load limits, corresponding to the primary stress limits. Table 5.11 shows the primary load limits given for each service condition. Rules for determining primary plus secondary stress limits and total stress limits were also developed. For details, see the Alternative Structural Evaluation Criteria for Class 1 Vessels Based on Elastic–Plastic Finite Element Analysis (Code Case NC-CC-005), JSME Codes for Nuclear Power Generation Facilities [20].

Table 5.11 Primary load limits by applying the limit analysis [20]

Service condition	Limit on primary load P_c^{*1}	How to calculate collapse load P_{cr} as allowable load		
		Calculation method	Yield point σ_y as elastic-perfectly plastic material	Temperature to determine S_m , S_y and S_u
Design condition		Asymptotic lower bound method*2 The twice slope method may be used.	$\sigma_y = S_m$ where S_m is the design stress intensity.	The maximum allowable working temperature (or design temperature)
Level C service condition			$\sigma_y = \min \left(S_y, \frac{2}{3} S_u \right)$ where S_y is the design yield point or strength. For austenitic stainless steels and high nickel alloys, $\sigma_y = 1.2 S_m$	The maximum temperature at events under the Level C service conditions
Level D service condition			$\sigma_y = \frac{2}{3} S_u$ For austenitic stainless steels and high nickel alloys, $\sigma_y = \min \left(2.4 S_m, \frac{2}{3} S_u \right)$	The maximum temperature at events under the Level D service conditions
Test condition			$\sigma_y = 0.9 S_y$	The maximum temperature under test conditions

[Notes]

*1 The load value given in the rectangular should be less than that in the circle.

*2 The collapse load to be obtained by this calculation method is the one that is determined to be converged in such a way that a load increment becomes sufficiently small as a load is appropriately and gradually increasing, by using the limit analysis (elastic plastic analysis assuming an elastic-perfectly plastic material).

5.9 The Practice of Structural Design

The concepts for assuring the structural integrity of components have been described in this chapter. It is desirable to understand the process and practice of structural design based on these concepts. First, the role of structural design in the entire design process and the stance required of structural designer are described, followed by an explanation of the criteria in the design codes. Also, the method for designing welds (or welded joints), which require particular care in designing components, is described.

Table 5.12 Stress classification and stress intensity limits in Design by Analysis approach based on elastic stress calculation for Class 1 vessels

		Stress classification			Secondary	Peak	Compressive*4
		Primary			Membrane + bending		(Compressive stress in cylindrical shell subjected to axial compressive load)
		General membrane P_m^{*1}	Local membrane P_L^{*1}	Bending P_b^{*1}	Q^{*1}	F^{*1}	
Service conditions	Design conditions	P_m → S_m	P_L → $1.5S_m$ P_L+P_b → $1.5S_m$ *7				
	Service limit for service conditions A and B				P_L+P_b+Q → $3S_m$ *2 P_L+P_b+Q+F (Total stress) → S_a *3		σ_c → min(S_m, B) B: the value B in the external pressure chart
	Service limit for service condition C	P_m → $kS_m, k=1.2$ *5 → min($S_y, \frac{2}{3}S_u$)*6	P_L → $1.5kS_m, k=1.2$ *5 → $1.5\min(S_y, \frac{2}{3}S_u)$ *6 P_L+P_b → $1.5kS_m, k=1.2$ *5 → $1.5\min(S_y, \frac{2}{3}S_u)$ *7		Evaluation not required		σ_c → 1.2min(S_m, B)
	Service limit for service condition D	P_m → min($2.4S_m, \frac{2}{3}S_u$)*5 → $\frac{2}{3}S_u$ *6	P_L → $1.5\min(2.4S_m, \frac{2}{3}S_u)$ *5 → $1.5 \times \frac{2}{3}S_u$ *6 P_L+P_b → $1.5\min(2.4S_m, \frac{2}{3}S_u)$ *7 → $1.5 \times \frac{2}{3}S_u$ *7		Evaluation not required		σ_c → 1.5min(S_m, B)
Limit for testing condition**8		P_m → $0.9S_y$	P_L → $1.5 \times 0.9S_y$ P_m+P_b → (1) for $P_m \leq \frac{2}{3}S_y$ $1.5 \times 0.9S_y$ (2) for $\frac{2}{3}S_y < P_m \leq 0.9S_y$ $2.15S_y - 1.2P_m$ *7		P_L+P_b+Q+F (Total stress) → S_a *9		

[Legend] : ○, ○ allowable value □ : calculated value

*1: Each of these symbols does not represent single quantities, but sets of six quantities representing six stress components σ_θ , σ_z , σ_r , $\tau_{\theta z}$, τ_{rz} and $\tau_{\theta r}$.

*2: A special rule called "simplified elastic-plastic analysis" is provided for exceeding $3S_m$. A rule called the "thermal ratchet limit (evaluation)" applicable to thermal stress under service conditions A and B is provided for the shell and nozzle of a vessel.

*3: S_a is obtained from the design fatigue diagram. The range of fluctuation is twice the amplitude and therefore the allowable stress intensity for the full range is $2S_a$. If there is more than one type of repetitive stress cycle, the cumulative usage factor U is limited to below 1.

*4: The allowable value for buckling under external pressure is obtained from diagrams A and B using the external pressure chart.

*5: Applicable to austenitic stainless steels and high-nickel alloys.

*6: Applicable to materials other than austenitic stainless steels and high-nickel alloys, such as carbon steels and low-alloy steels.

*7: As described elsewhere in this chapter, 1.5 is the shape factor of the cross section, which is rectangular in this case. For cross sections other than rectangles, the shape factor α for that shape should be used, instead of 1.5. $1.5S_m$ in the ASME Code is used here instead of $P_L + P_b \leq \alpha S_m$ in the JSME codes.

*8: The testing condition refers to the condition in which a pressure test, etc. are performed.

*9: Applicable when the number of pressure tests under testing condition exceeds 10.

5.9.1 *Structural Design and System Design*

There is no question that the most important in designing components is to assure structural integrity as the structural design, but that is not all. The structural design for specifying materials to be used, structural configurations, dimensions, assembling, etc., thermal design for developing heat and mass balance diagram, etc., control and instrumentation design, safety design, and so forth should be done to harmonize performance requirements and to optimize the plant design specifications by applying state-of-the art technologies. Further, for example, in order to assure structural integrity in a reasonable manner, requirements from the systems for achieving it need to be defined in a reasonable manner. What troubles the designers is that in the design stage prior to the operation of the plant, the frequency of occurrence of events and response to thermal transients must be determined. You will never know how often the event, such as loss of off-site power, occurs until the plant is operated. If the number of occurrences is set to a value realistic for the design life of a component, the plant must be shut down when the expected number of occurrences is reached. However, if an extremely large number of occurrences is assumed, the economy of the plant is compromised. Thus, the component design should not be positioned on the downstream side of the system design, but both design activities should collaborate spirally.

Generally, information regarding structural design is provided in the design specifications by the plant owner to the designers. However, what is important in achieving a reasonable design is that such information is not unilaterally provided. As shown in Fig. 5.27, the plant design process, particularly of Unit 1, proceeds from the basic planning of the layout of the buildings (or plot plan), plant systems and process design to determine system layout as well as heat and mass balance, control system and instrumentation design, safety design, and so on, in order to achieve the required performance and functions, with mutually consulting in a spiral way. On the basis of results of these designs, for example, major events to be evaluated should be determined, and then, the thermal response of the systems to the major events (the analysis for this purpose is called a system thermal transient analysis), the earthquake ground motion input into the buildings, and others are generated. Design specifications, and the loadings on the components and component requirements, which are determined from the thermal transient conditions (normally, in the form of thermal transient diagrams) and seismic design conditions obtained, are specified. In the stage of system design, the layout and main dimensions of the major components need to be tentatively determined. For example, a system thermal transient analysis requires information such as the heat capacity of the components. Therefore, the thickness of the major portions of components needs to be tentatively determined.

"Sizing" (calculation of the thickness of the major portions) in "Component design" in Fig. 5.27 is performed to tentatively determine the thickness. It should be noted here as an important point that we have the possibility of changing the dimensions such as wall thickness of major portions that are tentatively fixed for the system thermal transient analysis as a result of detailed structural design, typically

to meet stress limits for all service conditions. In such a case, trial and error will be needed to fix the dimensions and other design specifications. Time and manpower are consumed. A concept of “design conditions” for sizing becomes important, in particular for the Design by Analysis approach.

Components are designed based on this information. Major types and specifications of a component are determined to meet requirements on performance such as heat and mass balance, aseismic performance. Next, a detailed structural design is implemented on the basis of these design specifications. To meet the loading conditions and requirements, components are sized mainly to meet primary stress limits, and evaluated for primary plus secondary stress, fatigue damage and others (when the Design by Analysis approach is applied), and checked for structural integrity. Feasibility issues and issues requiring coordination such as interfaces with other components are identified in the process of component design, including thermal and structural design and auxiliary equipment design. As an example of such an issue, we can consider the case that too large seismic loads are expected to meet integrity of components as well as buildings. An idea to resolve this issue may be adoption of a base isolation system. These issues are fed back to the system design. The system design and the component design steps are repeated in a spiral manner until the optimal design is achieved.

In implementing design in a spiral manner, it is important to make an effort to achieve the optimal design with attention paid to the whole systems. It is not enough to focus only on structural design based on design specifications. It is quite important for members in charge of structural design to think hard and find the best design from the design specifications given by the owner, and further important for them to understand the overall design of the plant facility and to consider issues for improving to make the facility design better. It is important for the members in charge of system design to sincerely exchange ideas with members in charge of structural design. The author’s experience in developing an approach to assuring the structural integrity of a tokamak fusion system shows that it is important to develop a reasonable classification of components and events, requirements on component examination, and so forth, on the basis of considerations of the inherent safety of the fusion system, as well as based on the approach to assure structural integrity in a narrow sense as described up to Sect. 5.8, so as to always consider structural integrity in a broad sense. In some cases, structural designers should propose an approach to ensuring safety that they consider reasonable to system designers. To this end, they must be familiar with system design as well as fabrication and examination as suggested by the above discussion. New reasonable approaches often encounter resistance. It is needless to say that you need to study a broad range of subjects in depth.

5.9.2 Design Codes

In the previous sections, we learned the basic approach to assuring structural integrity and learned its historical background. This section briefly describes the process to

assure structural integrity based on Japanese design codes, or the flow of component sizing and stress evaluation. Stress limits obtained by the Design by Analysis approach based on elastic stress calculation described in Sect. 5.6 are described by the Hopper's diagram shown in Table 5.12. The table is based on the stress evaluation procedures (PVB-3100) for the design of Class 1 vessels in the design and construction rules for light water reactor components issued by the JSME (Japanese Society of Mechanical Engineers) [1] (partially based on the rules in the ASME Boiler and Pressure Vessel Code, Section III Division 1 Subsection NB, referred to as Ref. [2], taking into account the consistency with the descriptions in this chapter). The rules presented here are not all. For example, rules for special stress limits, such as on pure shear stress and bearing stress (note: the term "bearing" does not refer to "bearing" as a mechanical component), and limits on bolts and specifications on openings reinforcement are not included in this table. A special limit on compressive stress is shown in the right column in Table 5.12. It is not a limit on buckling under external pressure.

It should be noted here that Item [4] in Sect. 5.6.1 is described mainly based on Ref. [2] (or the ASME Boiler and Pressure Vessel Section III Division 1 Subsection NB). Consequently, allowable limits on primary stress intensities for service conditions C and D, etc. given in this Item may be a little different from Ref. [1] (or the JSME Codes for Nuclear Power Generation Facilities).

It is suggested that you check to see how what you have learned in the previous sections are applied to practical problems.

As you can see in the rules for primary stress limits in Table 5.12, there are cases where rules are available for design conditions but not for service conditions A and B. As mentioned in Item [4] in Sect. 5.6.1, this is due to the following reason. The maximum working temperature and pressure and design mechanical loads given as the design conditions for Class 1 components such as Class 1 vessels are determined on the basis of those for service condition A. They are set to values that exceed the maximum values for service condition A. However, service condition B should also be considered in determining the values. Therefore, limits are not explicitly placed on primary stress under service conditions A and B. The purpose of design conditions is in line with that of the Design by Formula approach and is to size a component or determine its thickness.

This is why a limit is placed only on primary stress. The limit on primary plus secondary stress due to cyclic loading and the limit on fatigue damage due to alternating peak stress intensity are not evaluated under design conditions. Therefore, these evaluations under service conditions A and B are required. There are no requirements for evaluation for cyclic loading under service conditions C and D. This is because the number of cycles of events postulated under these service conditions is small and evaluation for cyclic loading is considered to be unnecessary as long as the primary stress limits are satisfied. For loading with a large number of cycles, such as seismic loading described in Chap. 7, evaluation is performed as required. The details of the structural codes are slightly different from country to country. For example, the Japanese design codes for Class 1 vessels described above are based on the rules for Class 1 vessels of the ASME Boiler and Pressure Vessel Code Section

III Division 1 Subsection NB. However, they were streamlined based on the results of studies and have some unique rules.

If FEA described in Sect. 5.8 is applied, that the load P_C calculated by limit analysis is below the collapse load limit in Table 5.11 can be used as the criterion for Class 1 vessels alternative to the primary stress limit.

5.9.3 Weld Design

Welding is used to join materials such as metal plates into an assembly. The materials are melted and joined together at the interface. Welding is often performed at the plant construction site, resulting in a high probability of weld defects (see Chaps. 8 and 9 for the details of welding and weld examination). The following requirements apply to welds to prevent weld defects and achieve good welding quality [22].

- Welds must be continuous and not of abnormal configurations.
- It must be ensured by nondestructive testing that there is no possibility of cracking due to welding and there are no weld defects, such as lack of fusion, which are harmful in ensuring good welds.
- Welds must have adequate strength.
- Welding must be performed by the welding techniques, the procedures, welding equipment, qualified welders and others of which must be demonstrated to be appropriate beforehand by mechanical testing and other evaluation methodologies.

However, discontinuities in geometry are unavoidable to some extent in welds (or welded joints) including fillet welds. Also, there is the possibility that the properties of the weld, particularly the melted portion called weld metal, are different from those of the base metal before melting (therefore, those of the portion away from the weld). Such discontinuities in properties may be present. These discontinuities are collectively called structural discontinuities. It is necessary to assume that they are present in welds. A structural discontinuity leads to stress or strain concentration, resulting in reduced fatigue strength. The accuracy of detecting defects and structural discontinuities depends on the detection capability of the nondestructive examination method used.

It is not practical to separately develop a stress analysis model of a weld for structural design in which a structural discontinuity is simulated, taking into account properties specific to the weld. In the current design practice, the material properties of the base metal are used for those of the weld, and a so-called nominal-base structural geometry (the geometry assumed in design), which does not consider structural discontinuities present in the weld, is used in stress calculation. The stress in the weld is calculated from the thus obtained stress, taking into account the following factors, and evaluated to determine whether it is below the stress limit.

(1) Fatigue strength reduction factor (FSRF)

Design fatigue curves for base materials are applied to the evaluation of fatigue damage in a weld. The fact that a number of cycles to fatigue fracture of the weld is lower than that of the base metal (or, the fatigue fracture strength of weld decreases from that of the base metal.) is incorporated in fatigue damage evaluation as follow:

A model for elastic analysis to obtain an elastically calculated stress is generated by replacing the weld involving a structural discontinuity with the base metal involving no discontinuity. The elastically calculated stress obtained thus is denoted as S_a , while the stress or alternating peak stress intensity to calculate the real fatigue damage using the fatigue curve for the base metal is as S_a^* . The stress S_a^* in the weld may be expressed by the following equation in terms of the stress S_a :

$$S_a^* = S_a \times \text{FSRF} \quad (5.63)$$

Note that due to $\text{FSRF} \geq 1$, the allowable number of cycles of loadings on the weld decreases.

On the design fatigue curve of the base metal, the allowable number of cycles for the weld is N_a^* for S_a^* , when the corresponding allowable number of cycles for S_a is denoted as N_a , as schematically illustrated in Fig. 5.28. In this figure, it is shown how to determine the fatigue strength reduction factor, FSRF, from experimental data obtained from fatigue tests using welded joint specimens that are cut from welded joints. The FSRF is obtained by the following expression from the experimental data [23, 24]:

$$\text{FSRF} = \frac{S_{W0}}{S_W} \quad (5.64)$$

where S_{W0} is the stress amplitude for the plain specimen (or the non-welded specimen with the same configuration) for the specified number of cycles to fatigue fracture, and S_W the stress amplitude for the welded joint specimen for the specified number of cycles to fatigue fracture.

The FSRF obtained thus is used to evaluate the allowable number of cycles for the weld using the design fatigue curve of the base metal.

Regarding the fatigue test, pulsating tensile fatigue tests are preferred, because of avoiding buckling due to a compressive stress in an alternating fatigue test.

(2) Joint efficiency

For a welded joint, the regulation requires for pre-operational (or pre-service) examination of welds that a flaw or defect shall be removed when such a flaw or defect is detected by means of a nondestructive examination. The detail is discussed in Chap. 9. Since any nondestructive examination method has a detection limit below which it cannot detect the flaw or defect and the detection limit depends on the method, Further, a visual examination is permitted as weld examination. Therefore, the size of detectable flaw or defect depends on the weld examination method

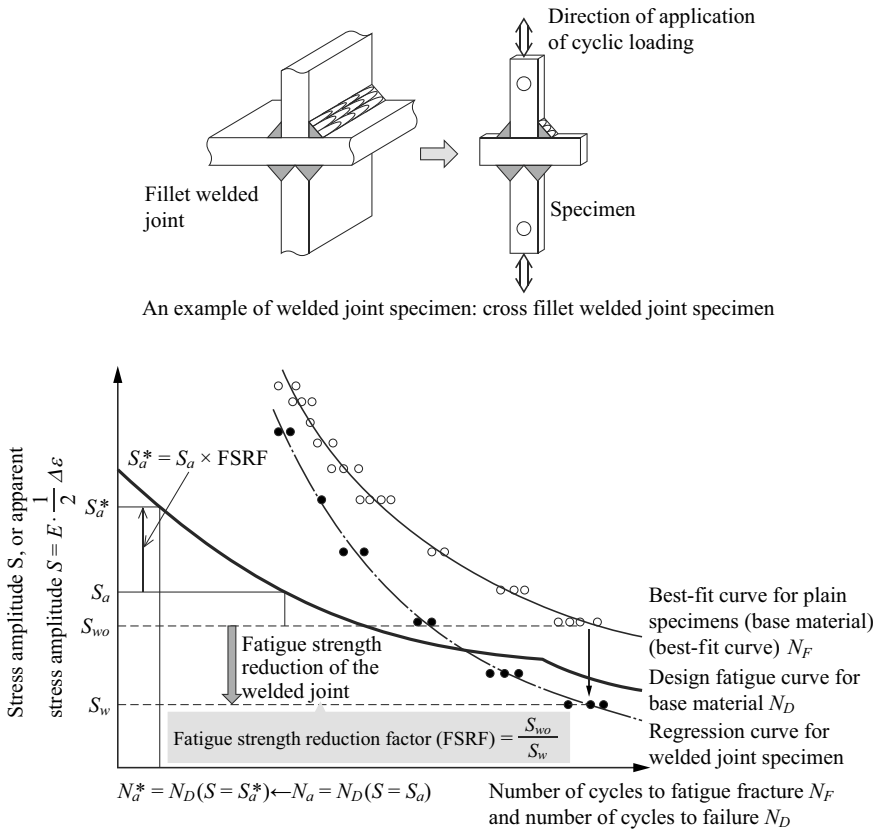


Fig. 5.28 Fatigue strength reduction factor of welded joint

required, resulting in a difference in quality level to be guaranteed. Structural codes and standards provide a reasonable rule, taking the detectable size into consideration.

As the reasonable rule, an allowable value of primary stress limit is reduced according to the detection capability of the nondestructive examination method used, by assuming that the welding was performed according to the specified welding procedures. The degree of this reduction is represented by joint efficiency.

It should be noted here that regarding a relation between defects detected and joint efficiency, defects existing just after welding remain unchanged, irrespective of nondestructive examination method applied. They depend primarily on a qualification level of welding procedure including welder and welding equipment. How large and many defects can be detected in a weld depends upon the examination method applied. This means that the quality level of weld varies according to not only the qualification level of welding procedure but also the examination method applied.

For Class 1 components, due to the very high level of quality required, nondestructive examination is required to ensure defect-free welds with an extremely high

level of quality. Because applying such an examination method that there is no need to assume reduced strength, the joint efficiency is not specified (in other words, the examination method to assure that the efficiency is 1.0 is required to apply).

(3) Consideration of residual stress

The region melted in welding is localized. The surrounding region does not melt, so that it pushes out the region that melted and expanded. Both the melted and surrounding regions are compressed. As a result, compressive plastic deformation remains in the surrounding region. Then, the melted region cools to solidify. The melted region contracts as it solidifies. Consequently, tensile residual stress occurs in both the weld metal and the surrounding region. Tensile residual stress is harmful in that it can reduce fatigue strength and cause stress corrosion cracking. Usually, post-weld heat treatment (PWHT) is performed to reduce the residual stress. However, evaluation of the level of the residual stress involves many uncertainties. To address the uncertainties, the maximum possible effect of mean stress is considered in fatigue evaluation, as mentioned in Item [2] (2) in Sect. 5.6.3.

In addition, welded joints are classified according to their importance. The importance of a joint depends on its location. Limits are placed on joint geometry and the level of examination is changed according to the importance of the joint. One of the reasons why the importance of the joint depends on its location is that the magnitude of the stress in a pressure-retaining component due to pressure varies with its direction. For example, in a cylindrical shell, the circumferential stress σ_θ is twice the axial stress σ_z . Therefore, an axial crack easily propagates and leads to fracture, compared to a circumferential crack. This is because the force to propagate and then open the crack is the force perpendicular to the surface of the crack, and the stress to open the axial crack is the circumferential stress. For this reason, a joint in the longitudinal direction of the cylindrical shell (in the direction of the weld line) is classified as a category A joint. A joint with the weld line in the circumferential direction is classified as a category B joint. Details are described in Chap. 9.

5.10 Components of Facilities for Coping with Severe Accidents, Etc.

5.10.1 Functions Required to Facilities for Coping with Severe Accidents, Etc.

In the sections so far, basic concepts for assuring the structural integrity of facilities that are required to function at the design basis events and relevant issues are discussed. These facilities are called the “facilities for design basis events” in this textbook, as mentioned in Chap. 1. In Japan, as the response to the Fukushima Daiichi nuclear accident, facilities for coping with severe accidents are required to install in addition to the facilities for design basis events. As the level 4 safety measures as

discussed in Chap. 1, we need to take measures against not only severe accidents at which a serious core damage occurs but also beyond design basis accidents with the possibility of escalating to severe accident (or beyond design basis accidents may lead to severe accident) but not severe accidents. Both categories of beyond design basis accidents are called the “severe accidents, etc.,” in this textbook. Consequently, as regulated facilities, we are required to install the facilities for coping with severe accidents, etc. as the level 4 safety measures.

Reasonable design concepts of components of the facilities for coping with severe accidents, etc. have not yet been established worldwide. For instance, for coping with severe accidents, etc. the following may be required to establish:

- (i) what kind of facility arrangement, configurations of systems to constitute the facility, arrangements of components to constitute each system and design concepts of components to constitute each system, and their functions and performance to be required to cope with the severe accidents, etc. are the best,
- (ii) what level of quality should be reasonably achieved of each component to attain the functions and performance required,
- (iii) what modes of failure are expected to occur at and after the severe accidents, etc., considering materials to be used and service conditions such as loading conditions, environmental conditions,
- (iv) what kinds of measures should be reasonably taken to prevent the modes of failure concerned,
- (v) what level of structural response should be reasonably kept, and
- (vi) what level of margin or design factor should be reasonably allowed to prevent each mode of failure, considering the severity of consequence, economical viewpoints, etc.

The NRA in Japan has provided the related regulations that are thought to be on the safe side. These regulations are given in the following.

In Japan, safety regulations on nuclear reactor facilities have been drastically changed and strengthened, reflecting the lessons learned from the Fukushima Daiichi nuclear accident. As a result, the following functions for the facilities for coping with severe accidents, etc. are prescribed in Article 37 of the NRA Ordinance on Standards for the Location, Structures and Equipment of Commercial Nuclear Power Reactors and Their Auxiliary Facilities.” Hereinafter, this is referred to as the “Ordinance on Standards for Location, Structures and Equipment” or simply the “Establishment Permit Standards.”

Subarticle 1: When an accident that may lead to a severe accident occurs, measures to prevent a severe damage of the reactor core should be taken for the nuclear power reactor facilities.

Subarticle 2: When a severe accident occurs, measures to prevent a failure of the reactor containment vessel and a release of an abnormal level of radioactive materials outside the facility factory and others should be taken for the nuclear power reactor.

Subarticle 3: When an accident that may lead to a severe accident occurs, measures to prevent a severe damage of the fuel assemblies and spent fuels in the spent fuel storage tank should be taken for the nuclear power reactor facilities.

Subarticle 4: When a severe accident occurs, measures to prevent a severe damage of fuel assemblies in the nuclear power reactor should be taken for the nuclear power reactor facilities.

It should be noted here that as mentioned in Sect. 1.2, the severe accident is defined in Article 4.1 of the “NRA Ordinance Concerning the Installation and Operation of Commercial Nuclear Power Reactor Facilities” (hereinafter, referred to as the “Ordinance on Installation and Operation”), as follows.

Item 1: Severe damage of the reactor core

Item 2: Severe damage of the fuel assemblies or spent fuels that are storage equipment of nuclear fuel materials

Facilities for coping with severe accidents, etc. include not only those that are installed solely for coping with severe accidents, etc. but also those in operation before a severe accident, etc. occurs (this means that these facilities are in operation at both of design basis events and severe accidents, etc.), and in addition those that are installed for coping with design basis events other than severe accidents, etc., namely not requiring the operation at severe accidents, etc. as a regulatory requirement. In other words, there are the facilities for design basis events that are also expected to function at severe accidents, etc. Figure 5.9 illustrates an image of correspondence between the above-mentioned facility classification and classification of accidents (or operating conditions) at which the facilities are required to function.

Table 5.13 shows what functions are required for coping with severe accidents, etc. and what types of facilities and equipment should be installed to achieve these functions, focusing on the reactor coolant pressure boundary and reactor containment vessel. In this table, the term “design basis extension” means that equipment and other components belonging to the facilities for design basis events are also designed to function at the severe accidents, etc. by simply extending their operating conditions.

5.10.2 Effectiveness Assessment of Equipment for Coping with Severe Accidents, Etc.

How the functions required as shown in Table 5.13 should be achieved for the equipment for severe accidents, etc. constituting the facilities for coping with severe accidents, etc. is prescribed in Article 54.1 of the “NRA Ordinance on Technical Standards for Commercial Nuclear Power Reactor Facilities” (hereinafter, referred to as “Ordinance on Technical Standards” as shown below.

Table 5.13 Functions to be required for facilities for coping with severe accidents, etc. (in particular, the reactor coolant pressure boundary and reactor containment vessel) and examples of facilities for achieving their functions

Article of the regulatory ordinance*	Content (required functions)	Facilities, etc. to implement the functions**	
		Facility and equipment	Their component classification
Article 37 Prevention and others of escalation of severe accidents, etc.	(Here, omitted because this is referred to in Section 5-10-1)	(Here, omitted)	
Article 39 Prevention of damage due to earthquake	1. The facilities for coping with severe accidents, etc. should meet the following requirements in accordance with facility classification listed below: (i) The facilities for coping with severe accidents, etc. in which permanent seismically important equipment to prevent severe accidents is installed (excluding specialized safety facilities). There should be no possibility of damaging the functions necessary to cope with accident that has the possibility of escalating to severe accidents against seismic forces due to the design basis earthquake ground motion. (ii) The facilities for coping with severe accidents, etc. in which permanent equipment for preventing severe accidents other than permanent seismically important equipment for preventing severe accidents is installed (excluding specialized safety facilities) They should withstand sufficiently against seismic forces calculated in accordance with rules of Article 4, Subarticle 2. (iii) The facilities for coping with severe accidents, etc. in which permanent equipment for mitigating severe accidents is installed (excluding specialized safety facilities) There should be no possibility of damaging functions necessary to cope with severe accident against seismic forces due to the design basis earthquake ground motion. (iv) The specialized safety facilities (Here, omitted) 2. (Here, omitted)	(Here, omitted)	
Article 40 Prevention of damage due to tsunami	(Here, omitted)		
Article 41 Prevention of damage due to fire	(Here, omitted)		
Article 42 The specialized safety facilities	(Here, omitted)		

(continued)

Table 5.13 (continued)

Article of the regulatory ordinance*	Content (required functions)	Facilities, etc. to implement the functions**	
		Facility and equipment	Their component classification
Article 43 Equipment for coping with severe accidents, etc.	1. The equipment for coping with severe accidents, etc. (Here, omitted) should meet the following requirements. (i) The equipment can effectively provide necessary functions to cope with severe accidents, etc. under the conditions of temperature, radiation, loading and in other service conditions that are caused by the assumed severe accidents, etc. (ii) The equipment should be reliably operated when the assumed severe accidents, etc. occur. (iii) The equipment should be tested or examined/inspected during operation of the nuclear power reactor or at shutdown in order to confirm its integrity and ability. (Here, the requirements following the above requirements are omitted.)	(Here, omitted)	
Article 44 Equipment for keeping the nuclear power reactor in subcriticality in case of failure of emergency shutdown	Nuclear power reactor facilities should have equipment necessary to maintain integrity of the reactor coolant pressure boundary and reactor containment vessel and to shift the reactor core into subcritical state in order to prevent a serious damage of the core even at an event with the possibility of escalating emergency shutdown failure of the nuclear power reactor at an abnormal (or anticipated) operational occurrence.	•Control rod driving piping (flow channel) for control rods emergency insertion by means of alternative control rods insertion function •Standby liquid control system	Both components in the left column are the "severe accidents, etc. class 2" (hereinafter, referred to as SA-2)
Article 45 Equipment for cooling the nuclear power reactor at an event of the reactor coolant pressure boundary at a high pressure	The nuclear power reactor facilities should have equipment necessary to cool the nuclear power reactor in order to prevent a serious damage of the reactor core, even at an event of loss of cooling function of the nuclear power reactor that the equipment for coping with design basis accidents has, when the reactor coolant pressure boundary is kept in a high pressure state.	•Alternative high pressure injection system •Reactor isolation cooling system (design basis extension) •High pressure core injection system (design basis extension)	The three systems in the left column : SA-2
Article 46 Equipment for depressurizing the reactor coolant pressure boundary	The nuclear power reactor facilities should have equipment necessary to depressurize the reactor coolant pressure boundary in order to prevent a serious damage of the reactor core and a failure of the reactor containment vessel, even at an event of loss of depressurization function of the nuclear power reactor that the equipment for coping with design basis accidents has.	•Relief safety valve •Depressurization by means of portable direct current (DC) power supply equipment	SA-2

(continued)

Table 5.13 (continued)

Article of the regulatory ordinance [*]	Content (required functions)	Facilities, etc. to implement the functions ^{**}	
		Facility and equipment	Their component classification
Article 47 Equipment for cooling the nuclear power reactor at an event of the reactor coolant pressure boundary at a low pressure	The nuclear power reactor facilities should have equipment necessary to cool the nuclear power reactor in order to prevent a serious damage of the reactor core and a failure of the reactor containment vessel even at an event of loss of cooling function of the nuclear power reactor that the equipment for coping with design basis accidents has when the reactor coolant pressure boundary is kept in a low pressure state.	•Alternative low pressure injection system •Low pressure injection system (design basis extension)	Both in the left column : SA-2
Article 48 Equipment for transferring heat to final heat sink	The nuclear power reactor facilities should have equipment necessary to transfer heat to the final heat sink in order to prevent a serious damage of the reactor core and a failure of the reactor containment vessel (limiting to a failure that occurs before a serious damage of the reactor core occurs) even at an event of loss of a function to transfer the heat to the final heat sink that the equipment for coping with design basis accidents has.	•Alternative reactor auxiliary cooling system	Permanent parts : SA-2
		•Reactor auxiliary cooling system (design basis extension)	Portable parts : SA-3
Article 49 Equipment for cooling in reactor containment vessel	1. The nuclear power reactor facilities should have equipment necessary to reduce the pressure and temperature in the reactor containment vessel in order to prevent a serious damage of the reactor core at an event of loss of cooling function in the reactor containment vessel that the equipment for coping with design basis accidents has. 2. The nuclear power reactor facilities should have equipment necessary to reduce the pressure, temperature and concentration of radioactive materials in order to prevent a failure of the reactor containment vessel at an event of developing serious damage of the reactor core.	•Alternative containment vessel spray cooling system (permanent) •Containment spray cooling system (design basis extension)	Both system components in the left column : SA-2
Article 50 Equipment for preventing failure due to overpressure of reactor containment vessel	1.The nuclear power reactor facilities should have equipment to reduce the pressure and temperature in the reactor containment vessel with keeping the reactor containment vessel boundary in order to prevent a failure of the reactor containment vessel due to overpressure when a serious damage of the reactor core occurs. (Here, the requirements following the above are omitted)	•Pressure relief device for reactor containment vessel	Permanent parts : SA-2
		•Alternative circulation cooling system	Portable parts : SA-3
Article 51 Equipment for cooling molten core at the bottom of reactor containment vessel	The nuclear power reactor facilities should have equipment necessary to cool the molten core that is fallen to the bottom of reactor containment vessel, in order to prevent a failure of the reactor containment vessel, when a serious damage of the reactor core occurs.	•Water injection system into the bottom of containment vessel	Permanent parts : SA-2 Portable parts : SA-3

(continued)

Table 5.13 (continued)

Article of the regulatory ordinance*	Content (required functions)	Facilities, etc. to implement the functions**	
		Facility and equipment	Their component classification
Article 52 Equipment for preventing failure of reactor containment vessel due to hydrogen explosion	The nuclear power reactor facilities should have equipment necessary to prevent a failure of the reactor containment vessel due to hydrogen explosion in a case when it is necessary to prevent a failure due to explosion caused by hydrogen (hereinafter, referred to as hydrogen explosion) in the reactor containment vessel when a serious damage of the reactor core occurs.	•Prevention of hydrogen explosion by means of inertization inside the reactor containment vessel: Inert gas system	(Facilities for design basis events)
		•Discharge of hydrogen gas and oxygen gas in the reactor containment vessel by means of containment vessel pressure relief device	Permanent parts : SA-2 Portable parts : SA-3
Article 53 Equipment for preventing damage of reactor building and others due to hydrogen explosion	The nuclear power reactor facilities should have equipment necessary to prevent a damage of the reactor building and others concerned due to the hydrogen explosion in a case when it is necessary to prevent a damage due to the hydrogen explosion of facilities to contain gaseous radioactive materials leaking from the reactor building and reactor containment vessel (hereinafter, these facilities are called the “reactor building and others”), when a serious damage of the reactor core occurs.	•Passive autocatalytic recombiner (PAR) •Instrumentation equipment for measuring hydrogen concentration in the reactor building	Not classified

* The “Ordinance on Standards for Location, Structures and Equipment of Commercial Nuclear Power Reactors and their Auxiliary Facilities.”

** The Nuclear Regulation Authority (NRA), “Common, Equipment for Coping with Severe Accidents, Etc., Kyo-ichi (Common No.1) : On Equipment Classification and Selection.” Regarding the Component Classification, refer to Table 5.14.

Item 1: Functions required to cope with severe accidents, etc. should be effectively performed at temperature, radiation, loads and under other service conditions experienced when an expected severe accident, etc. occurs.

Components and others constituting the equipment for coping with severe accidents, etc. are classified into several classes as shown in Table 5.14, in addition to the equipment classification as mentioned above, in order to satisfy the above-mentioned requirement. Materials used, structures and strengths of components and others for each class of the component classification should satisfy the requirements provided in Article 55 of the Ordinance on Technical Standards.” The components and others are defined as components and support structures (or supports), as mentioned earlier. The detailed prescription on structures and strengths is shown in Table 5.15. Severe

Table 5.14 Component classification of equipment for coping with severe accidents, etc. (referring to the “(NRA) Ordinance on Technical Standards for Commercial Nuclear Power Reactors and their Auxiliary Facilities”)

Component classification		Content
Primary item	Secondary item	
Class 1 component for severe accidents, etc. (or severe accidents, etc. class 1 component), abbreviated as SA-1	Vessel	The “severe accidents, etc. class 1 vessel,” “severe accidents, etc. class 1 piping,” “severe accidents, etc. class 1 pump,” or “severe accidents, etc. class 1 valve” is defined as vessel, piping, pump or valve belonging to the “equipment for coping with severe accidents, etc.” respectively (limited to those belonging to the “Specialized Safety Facilities”).
	Piping	
	Pump	
	Valve	
Class 2 component for severe accidents, etc. (or severe accidents, etc. class 2 component), abbreviated as SA-2	Vessel	The “severe accidents, etc. class 2 vessel,” “severe accidents, etc. class 2 piping,” “severe accidents, etc. class 2 pump,” or “severe accidents, etc. class 2 valve” is defined as permanent vessel, piping, pump or valve belonging to the “equipment for coping with severe accidents, etc.” respectively (excluding the Specialized Safety Facilities) (including permanent piping, valves, cables and the other components in the nuclear power reactor facility, to be connected to portable ones of equipment for coping with severe accidents, etc. (hereinafter, referred to as “portable equipment for coping with severe accidents, etc.”), among those connecting to the portable equipment for coping with severe accidents, etc. Hereinafter, those are referred to as “permanent equipment for coping with severe accidents, etc.”)
	Piping	
	Pump	
	Valve	
Class 3 component for severe accidents, etc. (or severe accidents, etc. class 3 component), abbreviated as SA-3	Vessel	The “severe accidents, etc. class 3 vessel,” “severe accidents, etc. class 3 piping,” “severe accidents, etc. class 3 pump,” or “severe accidents, etc. class 3 valve” is defined as vessel, piping, pump or valve of the portable equipment for coping with severe accidents, etc., respectively
	Piping	
	Pump	
	Valve	
Severe accidents, etc. class 1 support (or support structure)		Structure to support SA-1 components.
Severe accidents, etc. class 2 support (or support structure)		Structure to support SA-2 components.

accidents, etc. classes 1 to 3 components are required to maintain gross deformations within the elastic regime against loads specified as the design loadings (design condition loadings) at severe accidents, etc., and severe accidents, etc. classes 1 and 2 components and others (excluding valves and pumps) are required to prevent buckling. That is, components and others are required to have a wall thickness enough to meet the requirement on strength. Further, severe accidents, etc. classes 1 and 2 piping are required to prevent fatigue fracture as well as the expansion joints. For facilities for design basis events as well, the expansion joints and class 2 piping are required to implement a fatigue damage evaluation for preventing fatigue fracture even when the Design by Formula approach is applied. We can see from the above discussion and Table 5.15 that the severe accidents, etc. classes 1 and 2 components and others belonging to equipment for coping with severe accidents, etc. are required

Table 5.15 Rules on structures and strengths of components of equipment for coping with severe accidents, etc.^{*1}

Component classification	Type of component	Rules	Remarks
Severe accidents, etc. class 1 component and support		Item 4 of the Ordinance : Structures and strengths of the severe accidents, etc. class 1 components and severe accidents, etc. class 1 supports should satisfy the following requirements.	The rules in the left hand column are similar to those for class 2 components and supports of facilities for design basis events.
	Component	(i) Under design conditions ^{*2} , gross deformations should be limited within the elastic regime. (ii) Expansion joints should not cause any fatigue fracture for cyclic applications of stress for design condition loading. (iii) Class 1 piping (excluding the expansion joints) should not cause any fatigue fracture for design condition loading. (iv) Class 1 vessels and class 1 piping should not cause any buckling for design condition loading.	
	Support	Class 1 supports that are welded to the severe accidents, etc. class 1 components, and to cause damages of the severe accidents, etc. class 1 components due to their damages, should not cause ductile rupture and buckling.	
Severe accidents, etc. class 2 component and support		Item 5 of the Ordinance : Structures and strengths of the severe accidents, etc. class 2 components and severe accidents, etc. class 2 supports should satisfy the following requirements. For a component or a structure whose performance is identical to or greater than that to meet the following requirements, requirements to this component or structure are not limited to the following.	The rules in the left hand column are similar to those for the severe accidents, etc. class 1 components and supports.
	Component	(i) Under design conditions, gross deformations should be limited within the elastic regime. (ii) Expansion joints should not cause any fatigue fracture for cyclic applications of stress for design condition loading. (iii) Class 2 piping (excluding the expansion joints) should not cause any fatigue fracture for design condition loading. (iv) Class 2 vessels and class 2 piping should not cause any buckling for design condition loading.	
	Support	Class 2 supports that are welded to the severe accidents, etc. class 2 components, and to cause damages of the severe accidents, etc. class 2 components due to their damages, should not cause ductile rupture and buckling.	
Severe accidents, etc. class 3 component		Item 6 of the Ordinance : Gross deformations of structures and strengths of severe accidents, etc. class 3 components should be limited within the elastic regime for design condition loading.	The rules in the left hand column are similar to those for class 3 components and supports of facilities for design basis events, except no requirements to expansion joints and to prevent buckling.

[Notes]

*1: The above rules are specified in Article 55 of the “NRA Ordinance on Technical Standards for Commercial Nuclear Power Reactors and their Auxiliary Facilities.”

*2: The design conditions are defined as the conditions of loading in which the maximum allowable working pressure (or design pressure) and design mechanical loads are applied at the maximum allowable working temperature (design temperature). This definition is stipulated in Article 17, Subarticle 8 of the above Ordinance.

to assure the quality level identical to class 2 components and others of facilities for design basis events.

The following should be taken into consideration in applying the above-mentioned concepts to components and others belonging to equipment for coping with severe accidents, etc.:

- (a) The severe accidents, etc. class 2 components and others are permanent structures.

Taking it into consideration that these components and others have already been constructed and operated as a nuclear reactor facility component, if design conditions to be specified from loading conditions at severe accidents, etc. are encompassed by design conditions of the component of facilities for design basis events, it can be easily judged, on the basis of properly assessing the environmental conditions and others at severe accidents, etc., that the components and others belonging to the facilities for design basis events have structures and strengths enough to withstand against loadings at severe accidents, etc. However, we must consider several cases when the design conditions for components and others for coping with severe accidents, etc. may exceed those for components, and others for design basis events.

- (b) The Design by Analysis approach is applied to class 1 vessels, class 1 piping and boxes of class 1 valve belonging to facilities for design basis events. There are several cases when class 1 components and others of facilities for design basis events are classified as severe accidents, etc. class 2 components and others, because of their forming flow channels even at severe accidents, etc.

For the above-mentioned cases, first, confirming that existing components have structures and strengths sufficient to prevent failures to be expected to occur at severe accidents, etc., and then, it should be demonstrated that their gross distortions remain within elastic regime against design condition loadings that are specified for equipment for coping with severe accidents, etc.

In such a way, it can be demonstrated that the functions required are effectively performed. In the following, it is shown for a reactor containment vessel as an example how the functions required are achieved.

Effectiveness evaluation of containment vessel:

(1) Basic philosophy

The containment vessel is required to be functioned as a confinement for preventing undue release of radioactive materials outside the facility even at severe accidents, etc. in addition to design basis accidents like LOCA, and is classified as severe accidents, etc. class 2 vessel, as the final physical barrier. Consequently, the vessel is required to withstand occasional loading such as pressure, mechanical load and temperature that are produced not only at the design basis accident but also at severe accident, etc., and to maintain leak tightness as the confinement at both accidents. Thus, it is required to effectively achieve the following functions at severe accidents, etc.

- (a) To perform the pressure-retaining function, and therefore, to prevent expected modes of failure and to maintain gross deformations within the elastic regime, and
- (b) To perform the leak-tight function, and therefore, its deformations are limited to such low values that the gross distortion remains within the elastic regime.

Regarding the potential failure modes, Table 5.3 shows potential failure modes to be caused by a single and cyclic load applications in the LWR loading conditions. The modes of failure to be caused by cyclic load applications need not to consider at severe accidents, etc. because of a small number of cycles of applications. It has the possibility of brittle fracture, because its material is a carbon steel or a BCC (body-centered cubic) material. Since at severe accidents, etc., components in the reactor building are thought to be used at temperatures above the lowest allowable service temperature, the brittle fracture of the vessel does not need to consider its occurrence. Accordingly, ductile fracture and plastic collapse should be prevented as the potential failure modes to be caused at severe accidents, etc.

- (2) Prevention of ductile fracture and plastic collapse in order to effectively perform the pressure-retaining function

“Limiting pressure and temperature” are defined for the reactor containment vessel so as to assure its structural integrity at severe accidents, etc. These values are determined on the basis of experimental and numerical studies. The experimental studies include fracture experiments using one-tenth scale models [25].

The limiting pressure and temperature of the reactor containment vessel are established as follows:

Limiting pressure: twice the maximum service (or allowable working) pressure, or design pressure P_d , that is, $2P_d$, and

Limiting temperature: 200 °C

The prevention of both modes of failure at the limiting pressure and temperature can be discussed on the basis of the above-mentioned experimental results and other R&D results, and then it should be demonstrated that both modes of failure will be prevented with a sufficient margin for design condition loadings for equipment for coping with severe accidents, etc., and that deformations are limited to sufficiently low values.

- (3) Assurance of leak-tightness

At severe accidents, etc., a loss or degradation of leak tightness may occur at flanges of a pressure vessel or at penetrations such as cable penetration. Such a possibility might be suggested from experiences at the Fukushima Daiichi nuclear accident and laboratory experiments.

Assurance of leak tightness at the above-mentioned parts has been demonstrated under the deformation conditions of pressure vessels to be expected at the limiting temperature [26]. Consequently, it can be demonstrated to assure

leak tightness of the reactor containment vessel under the loading conditions at severe accidents, etc.

Hence, facilities for coping with severe accidents, etc. will effectively perform functions required even if severe accident, etc. may happen, and then prevent undue release of radioactive materials to the surrounding areas.

The above-mentioned approaches and the system of design codes to ensure structural integrity have been streamlined and updated to incorporate rapid technological advances. However, the current system of design codes was established based on experience. The Design by Analysis approach, which was standardized in the 1960s, is based on the then dominant stress calculation method. With 50 years since the standardization of the Design by Analysis approach, the infrastructure of design technology has undergone significant changes. For example, the FEM is now the dominant method of stress calculation. The system of design codes will be required to evolve into a system based on the latest technological infrastructure. The following systems of design codes are expected to be developed based on the latest technological infrastructure; research and development is being undertaken to achieve the expectation.

- (1) A system of design codes based on the probabilistic approach, such as a “system-based code”, to make it possible to reasonably assure structural integrity through the life cycle and quantitatively reflect the improvement in quality through technological advances, as described in Chap. 1
- (2) A “Design by Analysis” system that can be directly applied to an FEA or other stress analysis and relate the analysis results to failure modes, as described in Sect. 5.8.2
- (3) A system of design codes that can be applied to service conditions that cannot be covered by the current design codes due to aging, such as a significant change in material properties observed in regions damaged by intense neutron irradiation
- (4) Reasonable frameworks and detailed concepts for ensuring structural integrity of components belonging to equipment for coping with severe accidents, etc.

In order to continue to be an advanced-technology nation, Japan has to lead the world in research and development in this area and cutting-edge technology, propose innovative, reasonable structural codes incorporating such technology to the world, and thereby win the codes competition. Young researchers and designers are expected to play a central role in achieving this goal.

Exercises for This Chapter

1. There are two design approaches for critical reactor structures: the Design by Formula approach and the Design by Analysis approach.
 - (1) Briefly describe the characteristics of each design approach and the difference between the two.
 - (2) Briefly describe an example of the Design by Formula approach.
 - (3) Specifically describe the Design by Analysis approach.

2. Answer each of the following questions in 150 words or less.
 - (1) The test method for obtaining the design stress intensity, S_m , and the use of the design stress intensity
 - (2) The test method for obtaining the fatigue strength, S_a , and the use of the fatigue strength
3. Describe what you know about the design method for preventing fatigue failure of reactor structures.
4. Describe in an easy-to-understand manner how to determine a safety margin (design factor) in the structural design of the reactor, using, as an example, the failure modes of plastic collapse and fatigue failure.
5. The design of nuclear structural components uses the concept of stress classification to reasonably prevent failure events. Answer the following questions:
 - (1) Describe stress classification using specific examples.
 - (2) Describe the failure modes to be considered in structural design using a specific example.
 - (3) Describe the relationship between the stress categories and the failure modes.
6. Answer the following questions on the criteria for preventing ductile fracture and plastic collapse used in the design of nuclear structural components:
 - (1) Describe ductile fracture and plastic collapse.
 - (2) Describe the method for determining the allowable stress, using a specific example.
 - (3) Describe the method for determining the design factor using a specific example.
7. Nuclear structural components such as the reactor pressure vessel are welded structures and designed taking into account their characteristics. Answer the following questions:
 - (1) Describe the characteristics of welded structures.
 - (2) Describe the measures for preventing failure of welded structures taken in each of the design, manufacturing, installation, and examination processes using a specific example.
8. In component design, if the value obtained by the Design by Analysis approach is used as the allowable stress, the Design by Formula approach is not allowed to be applied to thickness calculation and stress evaluation. Explain why.
9. Assumptions are made and conditions are specified so that the equations to obtain stress limits are conservative. Describe what you know about the following stress limits:

- (1) Primary stress limit
- (2) Thermal ratchet limit
- (3) Fatigue limit
- (4) Buckling limit

10. (1) What portion of a structure is susceptible to fatigue failure? Look at field data on fatigue failure of light water reactor structures and answer the question.
- (2) Describe the basic principle of the prevention of fatigue failure applied to critical light water reactor structures, using the following words:

traditional endurance limit design, the Design by Analysis approach, high-cycle fatigue, low-cycle fatigue, cumulative fatigue damage, Miner's rule, fatigue test data, design factor (safety factor), mean stress, modified Goodman's diagram.

11. Select the failure modes that can occur from use in the non-creep region and is caused by defects such as cracks. Then, describe the measures taken to prevent each of them in terms of the prevention of the occurrence and propagation of defects in 100 words or less.
12. Briefly describe the following terms.
 - (1) Stress classification
 - (2) Thermal ratchet
 - (3) Fatigue strength reduction factor
 - (4) Limit analysis
 - (5) Stress intensity
 - (6) Limit load
 - (7) Weld joint efficiency
 - (8) Plastic collapse
 - (9) Elastic-perfectly plastic approximation
 - (10) Thermal stress ratchet
 - (11) Equivalent stress and equivalent strain
 - (12) Design fatigue curve
 - (13) Secondary stress

13. Answer the following questions on structural design of a nuclear power plant
 - (1) Describe reason why allowable limits on membrane plus bending stress are different from those on membrane stress
 - (2) Describe reason why a coefficient for yield stress for determining design stress intensity is different between for ferritic steels and for austenitic stainless steels.
 - (3) Describe reason why an allowable stress value for secondary stress is larger than that for primary stress.
 - (4) Describe reason why regarding fatigue evaluation, a simplified elastic plastic analysis is applied to a high stress

14. Describe what you know about the following items regarding evaluation on class 1 components in structural design standards for nuclear power plants (such as JSME Codes for Nuclear Power Generation Facilities, Rules on Design and Construction for Nuclear Power Plant)
- (1) Reason why allowable limit values on primary membrane plus bending stress are higher than those on general primary membrane stress.
 - (2) Reason why higher allowable values on primary plus secondary stress range are permitted than those on primary stress alone.
 - (3) Reason why for normal service components, allowable limit values on primary stress under operating condition IV are higher than under operating condition III.
 - (4) Reason why for fatigue analysis, an amplification factor is specified to multiply by peak stress intensity, as needed.
 - (5) Reason why the stress intensity according to the maximum shear stress criterion is basically used for integrity evaluation but not the principal stress.
15. The following sentences explain a general procedure for implementing structural design of LWR class 1 vessels according to the JSME (Japan Society of Mechanical Engineers) Code for Nuclear Power Generation Facilities and others. Answer adequate words or numerical values in the parentheses in the following sentences with the numbers. The same words or numerical values shall be used in the parentheses with the same circled figure.

[An example of the answer] (2) Tokyo.

First, make an evaluation of (2) which is produced by (1), external loads and others at a normal operation, and is not relaxed (or reduced) due to plastic deformation. (4) calculated according to (3) theory from the average value across the thickness of a section of the individual components of (2) which occurs in portions remote from structural discontinuities under design conditions shall be limited to the design (4) established for each material. The design (4) shall be determined as the lesser of (5) of the design yield strength (or, yield stress, yield point) (for austenitic stainless steels and high-nickel alloys which have larger (6), 0.9), and (7) of the design tensile strength.

Further, considering that under bending stress, (8) is higher than the load to initiate yielding, (4) after linearizing the stress distribution across the wall thickness is required to be limited to a constant times the design (4) (for solid rectangular sections, the value of constant is 1.5).

Next, an evaluation of (10) as represented by a thermal stress shall be made. Since (10) has a characteristic of displacement-controlled (or deformation-controlled) stress that is relaxed with increasing plastic deformation, and has a low possibility to cause fracture by itself alone, a higher allowable value is established than (2). The maximum allowable range of thermal stress is limited depending on (2), for preventing (11) that may occur under certain combinations of (2) and thermal stress.

Further, an evaluation of (13) is made for (10) intensity which includes (12), or a non-linear component of stress distribution. For (13) evaluation, it is required that (15) obtained by means of inserting the stress amplitude, which modifies the amplitude of (12) intensity by considering a strain amplification due to plastic deformation, into (14), shall not be less than the number of cycles to be expected for a real component. If there are multiple loading conditions, for each loading condition, calculate the ratio of expected number of cycles to (15), and then (16) calculated by summing up them for all the loading conditions shall not exceed 1.0.

In addition to the above, in order to demonstrate that for a reactor pressure vessel, (18) does not occur even under thermal shock when (17) actuates, (20) -based evaluation is made on the assumption that there exists (19).

16. The following are extracts from the “terms relating to stress analysis,” of the JSME Codes for Nuclear Power Generation Facilities. Answer adequate words in the parentheses in the sentences with the numbers. The same words shall be used in the parentheses with the same circled figure.

[An example of the answer] (11) Tokyo.

- (1) is the component of normal stress (normal (or perpendicular) to the plane of reference) that is equal to the average value of normal stress across the thickness of the section under consideration.
- (2) is the variable component of normal stress, as varying from the average value.
- (3) is any normal stress or (4) developed by an imposed loading that is necessary to satisfy the law of equilibrium of external and internal forces and moments.
- (5) is the normal stress or (4) developed by the constraint of adjacent material or by self-constraint of the structure.
- (6) is an increment of stress that is additive to (3) plus (5) by reason of local discontinuities or local thermal stress including the effects, if any, of stress concentrations.
- (7) is 1) resulting from application of internal pressure and mechanical loading, remote from structural discontinuities and stress concentrations.
- (8) is the stress resulting from restraint of thermally induced free end displacement of a piping system due to support points.
- (9) is the stress intensity of the combination of (3), (5) and (6).
- (10) is the summation the ratios for all types of stress cycles that for each type of stress cycle is defined as the ratio of the specified number of times that each type of stress cycle will be repeated during the life of the component to the maximum number of repetitions which would be allowable if this type of cycle were the only one acting, determined for the alternating (9).

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Chapter 5 focuses on the understanding of concepts. The following books are recommended if you wish to learn more about these concepts and other topics.

Further Readings

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Chapter 6

Structural Design for Fast Reactors



The structural design methods for LWRs described in Chap. 5 are applicable to the components subjected to low temperatures for which creep effects can be neglected. On the other hand, the operating temperatures of fast reactors include the range potential creep deformation. In addition, elevated temperatures imply that a larger temperature gradient exists with structures at low temperature, causing more severe thermal stress. Consequently, different failure modes than those affecting the LWRs have to be taken into account when designing fast reactor components subjected to high temperatures.

This chapter will cover these specific considerations by describing the design flow and design methods that are specific to fast reactors. An emphasis will be put on the design methods that aim at preventing the failure modes specific to fast reactors, such as deformations due to cyclic thermal stress and creep-fatigue damage.

6.1 Failure Modes to Be Considered in Fast Reactors

6.1.1 Characteristics of Fast Reactor Operating Conditions

In fast reactors (FRs), liquid-metal sodium is typically used as coolant to remove heat from the core, as it provides low neutron moderation and high heat conductivity. Since sodium has a high boiling point, fast reactors are operated at high temperatures without pressurization, which are advantageous in terms of thermal efficiency. An example system configuration of loop-type fast reactors, such as Monju, is displayed in Fig. 6.1. The sodium in the primary cooling system flows out from the upper part of the reactor vessel after being heated by the core. Then, it flows through the piping system to the intermediate heat exchanger, and returns to the lower part of the reactor vessel after having transferred its heat to the sodium in the secondary cooling system. The heated secondary sodium is supplied to the steam generator, in

which heat is transferred to the water-steam system connected to the turbine which generates electricity. In this regard, the reactor vessel as well as the components and piping in the primary and secondary cooling systems are designed with different specifications than those for the LWR as listed in Table 6.1, specifically due to the use of sodium as the coolant and the high operating temperatures. These particularities influence the loading conditions, and therefore the overall structural design for fast reactors as shown in Fig. 6.2.

The first distinctive feature of the fast reactor, as opposed to the LWR, is that creep characteristics of material must be taken into account since the operating temperature is high (normally, the reactor outlet temperature ranges from 500 to 550 °C). The second distinctive feature is that the thermal stress becomes the dominant load since the difference between the core inlet and outlet temperatures is large. In addition,

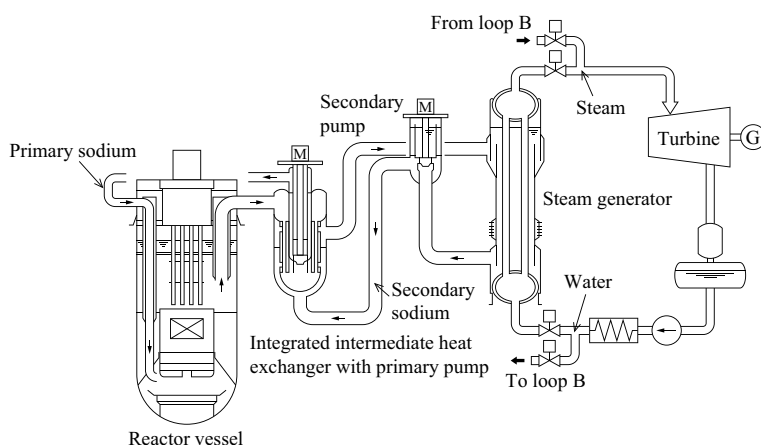


Fig. 6.1 Example of (loop-type) fast reactor system configuration

Table 6.1 Comparison of design specifications between LWR and FR

	LWR	FR (sodium-cooled reactor)
Reactor outlet temperature	PWR: approx. 325 °C BWR: approx. 286 °C	Monju: 529 °C Demonstration reactor: approx. 550 °C
Inlet/outlet temperature difference	PWR: approx. 35 °C BWR: approx. 70 °C	Monju: 132 °C Demonstration reactor: approx. 150 °C
Operating pressure	PWR: approx. 16 MPa BWR: approx. 7 MPa	Not exceeding 1 MPa
Reactor vessel wall thickness (Radius/wall thickness ratio: R/t)	PWR: approx. 220 mm, R/t $\approx$ 10 BWR: approx. 160 mm, R/t $\approx$ 20	Monju: 50 mm, R/t $\approx$ 70 Demonstration reactor: 50 mm, R/t $\approx$ 100

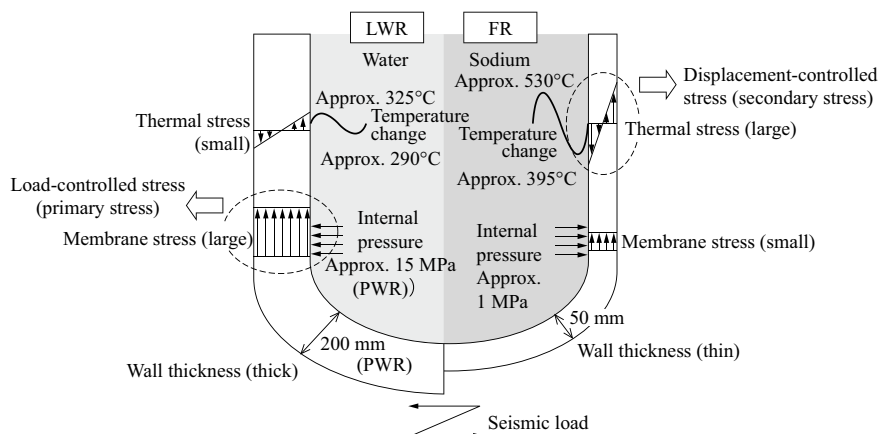


Fig. 6.2 Comparison of loading conditions between LWR and FR

transient variations in coolant temperature, caused by changes in plant operating conditions, are rapidly transferred to the components, with a large range of variation. These are mainly due to the thermal properties of sodium, which have a larger heat transfer coefficient and a smaller heat capacity than water. Nevertheless, since the thermal stress is a displacement-controlled stress, it is unlikely to cause a sudden rupture by a single load, and accordingly, the main failure modes to be considered are (creep-)fatigue cracks and excessive deformation due to cyclic loading. The third feature is that the internal pressure applied to vessels and piping is small, approximately 1 MPa even in high pressure region. Indeed, there is no need to pressurize sodium as its boiling point is approximately 880 °C. Therefore, small stress is caused by internal pressure. The fourth feature is that a particular attention has to be given to seismic designs and buckling failures since thin-walled structures have been advantageously adopted to reduce the effect of thermal stress, while bearing small risk from the low internal pressure. For these reasons, the “Codes for Nuclear Power Generation Facilities: Rules on Design and Construction for Nuclear Power Plants” of the Japan Society of Mechanical Engineers requires application of Edition I—Rules for Light Water Reactors (hereinafter, “LWR Rule”) [1]—at temperatures below or equal to 375 °C (for ferritic steel) or below or equal to 425 °C (for austenitic stainless steel) above which creep effects have to be taken into account. Above these temperature, application of Edition II—Rules for Fast Reactors (hereinafter, “Design rule”) [2] is required. The latter rules require that design activities consider the failure modes listed in Table 6.2. Austenitic stainless steels (304SS and 321SS for Monju) that have good compatibility with sodium and superior strength characteristics against high temperature, and are therefore employed as the main materials for the reactor vessel, the cooling system components, and the piping system of the fast reactor system (except the steam generator), as illustrated in Fig. 6.1. For the steam generator, ferritic steel (2 1/4Cr-1Mo steel for Monju) is used as it possesses advantageous properties in terms of corrosion resistance in a water environment. For future fast reactors, a

Table 6.2 Possible failure modes in the fast reactor design rule

In common with LWR (at low temperatures: non-creep range)	Ductile fracture, plastic collapse
	Excessive plastic deformation
	Fatigue damage
	Elastic–plastic buckling
Specific to FR (at high temperatures: creep range)	Creep rupture
	Excessive creep deformation
	Creep-fatigue damage
	Creep buckling

wide range of high-chromium ferritic steels for the cooling system components is expected to be applied, since they are less affected by thermal stress due to their excellent high-temperature strength and their small coefficient of thermal expansion.

6.1.2 Structural Design Flow for Fast Reactor Components

The “design by analysis” concept, based on detailed stress analysis, is applied to Class 1 and Class 2 components of fast reactors. Particularly, the design analysis focuses on failures due to thermal stress. This approach is generally stated as follows (Chap. 5): “the detailed assessment of the structural integrity should be achieved by performing analyses that consider all the possible failure modes, as well as provide the design procedures associated to those failure modes.”

Figure 6.3 described a particular schematic flow considering thermal loads. At the upstream design stage, the design of the plant systems is carried out (step 1). In this first step, the arrangement, size, and dimensions of components are temporarily determined while maintaining consistency with the plant concept, core design, piping type, system design (pipe size, liquid level, etc.) shielding design, refueling method, in-core instrumentation, as well as the ability to correctly manufacture, assemble, inspect, and maintain the design. Next, the events relevant to the system design step are determined (step 2). Based on the above conditions, the design of each component is carried out (step 3). The material is selected based on the conditions of temperature, neutron irradiation, etc.; the head shape and wall thicknesses are decided from the internal pressure and earthquake resistance. Then, the structural design is carried out in detail considering the arrangement of the components. At step 4, the flow conditions for the components are determined based on the component design. The plant dynamic analysis and in-core flow analysis are performed to obtain the in-core temperature and flow rate distributions, heat transfer coefficients, etc. From these results, the loading conditions of the structures are assumed (step 5). The loads considered include the internal pressure, self-weight, thermal transient and seismic loads, pipe reaction force, thermal boundary conditions, and gamma heating. Subsequently, failure modes are assumed (step 6). Consideration is also

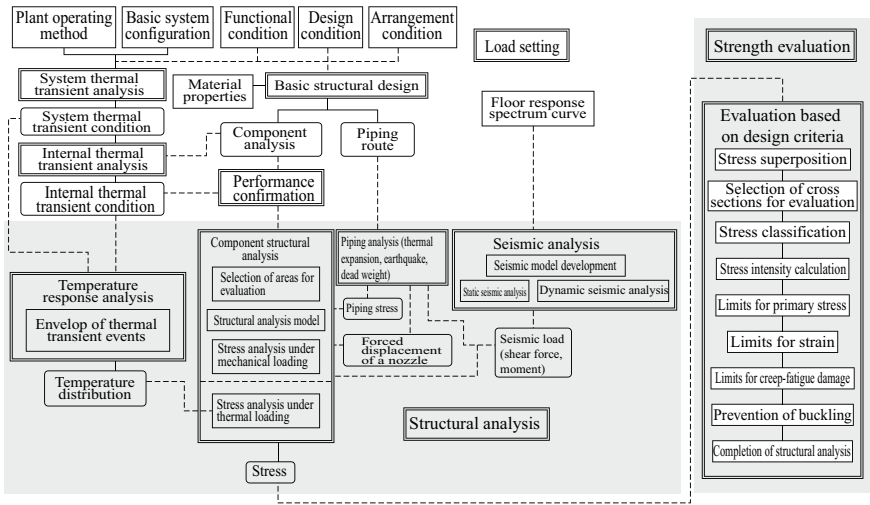


Fig. 6.3 Schematic flow diagram for component structural design with the thermal load considered

given to the environmental effects (sodium, oxidation, irradiation, etc.), weld joint, manufacturing, installation accuracy, etc. in addition to the loading conditions. Analysis methods are selected to perform structural analysis (step 7). FEM elastic analysis is the most applied method. Inelastic analysis and other approaches are carried out too when needed. In the final step, the strength of the structures is evaluated (step 8). The failure modes described in the design standards are evaluated using the specified failure criteria and safety margin. The main items of the above-mentioned design procedure are further described in the following sections.

6.2 Thermal Load Setting and Thermal Stress Analysis

6.2.1 Types of Thermal Load and Their Settings

As described in the previous section, various types of loads, including the internal pressure, self-weight, thermal transient and seismic loads, are considered for component design. Among these loads, additional attention is held on thermal stress caused by varying coolant temperature, as it is the dominant stress in fast reactors. It also involves both thermal hydraulic and structural aspects within complicated features.

The fast reactor systems are subject to steady-state thermal stress and thermal transient stress due to occurring variations of the system temperature, depending on the operational conditions. Typical thermal loads encountered in the fast reactors are illustrated in Fig. 6.4 along with explanatory comments on the mechanisms inducing

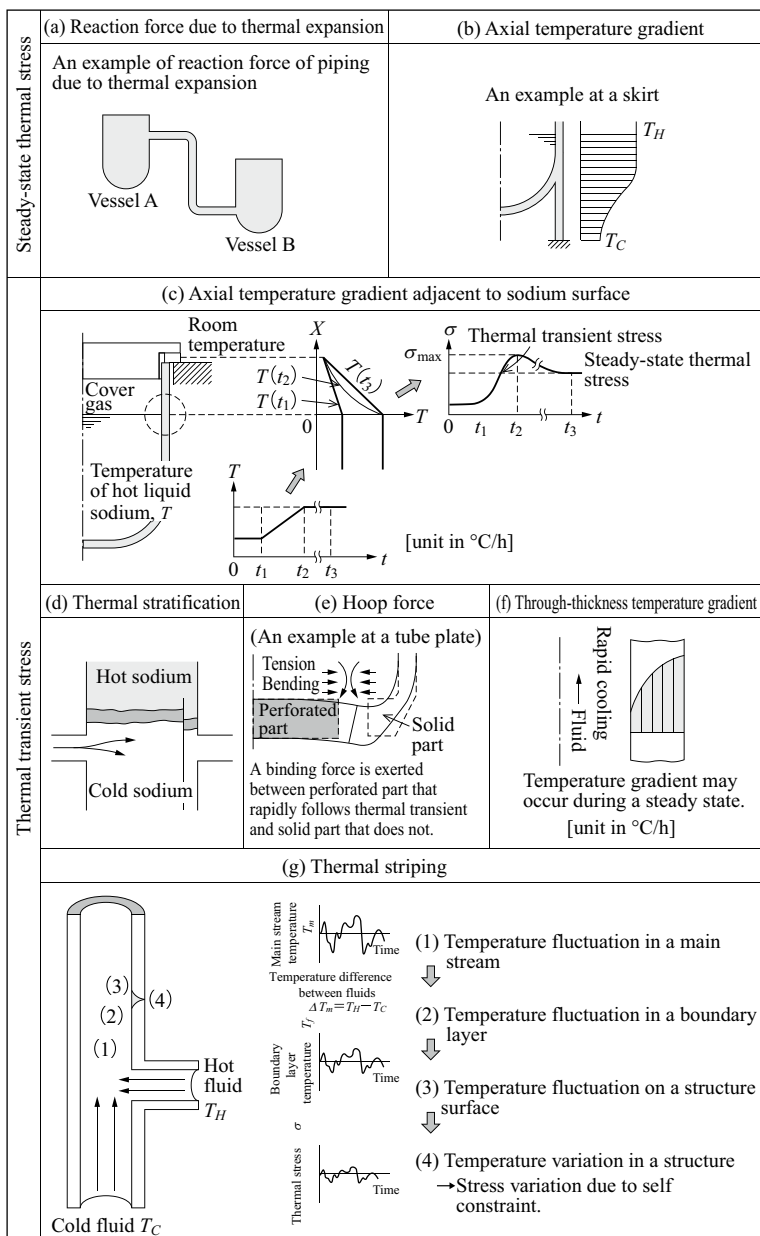


Fig. 6.4 Typical thermal loads in the fast reactor

thermal stresses. The analytical and numerical thermal stress evaluations will be explained in following Sects. 6.2.2 and 6.2.3.

Figure 6.4 shows thermal stress (a) as a steady-state reaction force exerted on a pipe connecting two vessels which are immovably supported on a concrete floor of a building. The stress is generated by the constraint of thermal expansion caused by the temperature rise during operation.

In Fig. 6.4b, thermal stress is pictured as a steady-state stress caused by the difference in thermal expansion between the support structure fixed to a concrete floor and the vessel. The support structure is maintained at low temperature while the vessel is subjected to high temperatures during operations, thus causing the temperature gradient.

Figure 6.4c describes a thermal stress caused by a vertical temperature gradient (in the axial direction) in the vessel wall, caused by the existence of free surface of sodium inside the vessel. This loading occurs in both transient and steady-state conditions. This type of thermal stress has a long response time constant as it depends on the temperature distribution from the liquid surface to the top of the vessel (long distance). As a consequence, it becomes a limiting factor even for mild transient operating conditions that take more than several hours, as observed in startup and shutdown conditions of a plant.

Thermal stress (d) displayed in Fig. 6.4 is generated by a phenomenon that can occur when cold sodium flows into a hot sodium region at a low flow rate. This phenomenon is called thermal stratification and it causes a thermal stress on the contacting vessel wall due to a vertical temperature gradient like in the previous example (c). This type of temperature gradient in the vertical direction establishes itself for a certain period, under specific conditions such as when hot and cold sodium cannot be easily mixed due to the balance between buoyant forces and inertial forces, resulting from the difference of density between hot and cold sodium.

Figure 6.4e shows thermal stress as a binding force caused by mutual constraint between two connected parts subjected to thermal expansions at different speed. The first part rapidly follows the temperature variations while the other part can only follow it slowly. A typical example of such phenomenon occurs in heat exchanger tube sheets, which consists in a perforated part, through which coolant flows, and a solid rim surrounding the structure.

Thermal stress (f) in Fig. 6.4 is a stress caused by a through-thickness temperature gradient caused by the same mechanism as the one described in (e). When the coolant flows inside a pipe while experiencing rapid temperature variations, the thermal expansion response of the inner surface occurs rapidly, while the outer surface responds slowly. Such gap causes a temperature difference within the pipe wall, thus inducing thermal stress. This type of stress becomes a problem when the coolant temperature change is so rapid that the difference in temperature responses between the inner and outer surfaces reaches a significant level. The thermal transient stress caused by the coolant temperature variation becomes the main load for plant components. To estimate such type of thermal transient load, it becomes necessary to perform both plant-wide thermal transient analysis as well as detailed thermal hydraulic analysis of each component. In addition, due considerations must be taken regarding several factors influencing the thermal hydraulic analysis, while they remain mostly uncertain during the design phase. Figure 6.5 shows an example

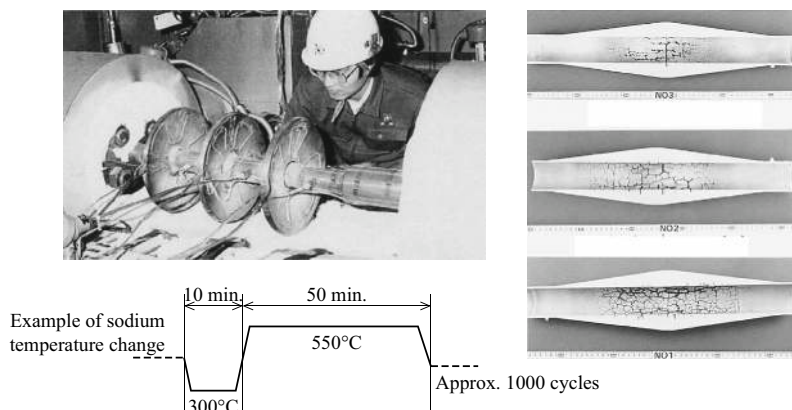


Fig. 6.5 Creep-fatigue cracks caused by cyclic thermal stress

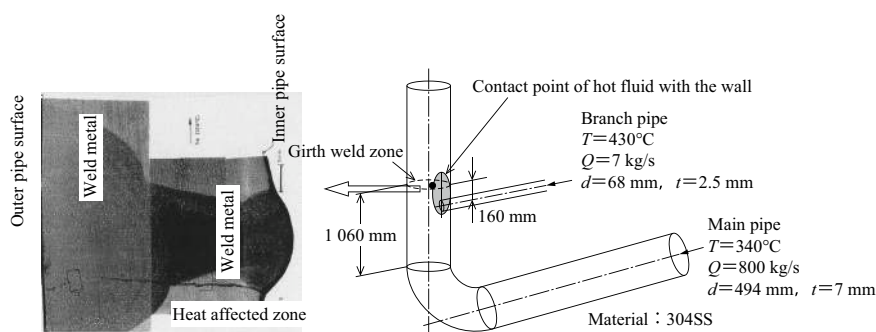


Fig. 6.6 An example of high cycle thermal fatigue cracks caused by fluid temperature fluctuation

of creep-fatigue cracks caused by applying a cyclic thermal transient load. In order to prevent this type of failure, it is necessary to appropriately evaluate the thermal stress.

Thermal stress (g) in Fig. 6.4 shows a phenomenon called thermal striping. Random fluid temperature fluctuations at relatively high frequency are likely to occur in mixing regions of hot and cold fluids, leading to high-cycle thermal fatigue in the neighboring structure due to continuous variation of the through-wall temperature gradient. Similar types of failure caused by this mechanism were observed in LWRs. In recent years, several failure cases caused by thermal striping have been reported in both LWRs and fast reactors in Japan and overseas. Figure 6.6 displays an example of high cycle thermal fatigue cracks observed at the weld region adjacent to a pipe junction where hot and cold sodium merge, in the French prototype fast reactor Phenix. The thermal striping phenomenon follows a specific process, described in the next paragraph.

An evaluation guideline based on the failure mechanism [3] has been developed. When hot and cold fluids are mixed, temperature fluctuation occurs in the mainstream (step 1), leading to fluid temperature fluctuation within a boundary layer via heat transfer in the fluid (step 2). Next, this temperature fluctuation is transferred to the inner surface of the structure (step 3). Subsequently, temperature fluctuation appears in the structure due to thermal conduction, leading to thermal stress as local thermal expansions are naturally constrained within the structure (step 4). When such thermal stress is repeated many times with sufficiently high stress amplitude, high cycle thermal fatigue cracks initiate on the surface of the structure (step 5). Continuous application of the thermal load would then lead to crack propagation (step 6).

6.2.2 *Simplified Method for Calculating Thermal Stress Based on the Thermoelastic Theory*

An example of analysis based on the thermoelastic theory is described for better understanding on how thermal stress would occur. In general, thermal stress calculations for arbitrarily shaped structures are performed with numerical simulation tools, such as the finite element method, regardless of the elastic or inelastic characteristic of the analysis. Despite so, simplified calculation methods are often useful for estimating the validity or for understanding the tendency of the numerical calculation results.

[1] **Thermal stress in a cylinder due to a through-wall temperature gradient**

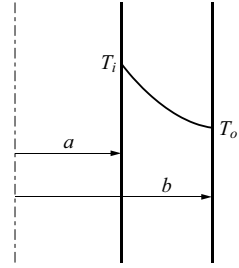
The solution for the thermal stress in a cylinder due to a through-wall temperature gradient is useful in estimating the thermal stress shown in (f) or (g) in Fig. 6.4. When a long cylinder of inner and outer radii of a and b , fixed at both ends, is subjected to a through-wall temperature distribution T (Fig. 6.7), the solution of the thermal stress (triaxial stress) problem is obtained from the thick wall cylinder theory as follows (e.g., Ref. [4]):

$$\sigma_r = \frac{\alpha E}{1 - \nu} \left(\frac{1}{r^2} \frac{r^2 - a^2}{b^2 - a^2} \int_a^b T r dr - \frac{1}{r^2} \int_a^r T r dr \right) \quad (6.1)$$

$$\sigma_\theta = \frac{\alpha E}{1 - \nu} \left(\frac{1}{r^2} \frac{r^2 + a^2}{b^2 - a^2} \int_a^b T r dr + \frac{1}{r^2} \int_a^r T r dr - T \right) \quad (6.2)$$

$$\sigma_z = \frac{\alpha E}{1 - \nu} \left(\frac{2}{b^2 - a^2} \int_a^b T r dr - T \right) \quad (6.3)$$

Fig. 6.7 Cylinder with a through-wall temperature gradient



where E is Young's modulus, α is linear expansion coefficient, ν is Poisson's ratio, T is temperature, and r is radius.

Specifically, in a steady-state thermal hydraulic condition where the temperatures at the inner and outer surfaces are maintained at constant values T_i and T_o , the temperature distribution is given by

$$T = T_i - \frac{\ln(r/a)}{\ln(b/a)}(T_i - T_o) \quad (6.4)$$

The stresses under this condition are expressed by the following equations.

$$\sigma_r = \frac{\alpha E(T_i - T_o)}{2(1 - \nu) \ln(b/a)} \left\{ -\ln \frac{b}{r} - \frac{a^2}{b^2 - a^2} \left(1 - \frac{b^2}{r^2} \right) \ln \frac{b}{a} \right\} \quad (6.5)$$

$$\sigma_\theta = \frac{\alpha E(T_i - T_o)}{2(1 - \nu) \ln(b/a)} \left\{ 1 - \ln \frac{b}{r} - \frac{a^2}{b^2 - a^2} \left(1 + \frac{b^2}{r^2} \right) \ln \frac{b}{a} \right\} \quad (6.6)$$

$$\sigma_z = \frac{\alpha E(T_i - T_o)}{2(1 - \nu) \ln(b/a)} \left\{ 1 - 2 \ln \frac{b}{r} - \frac{2a^2}{b^2 - a^2} \ln \frac{b}{a} \right\} \quad (6.7)$$

When $T_i < T_o$, σ_θ becomes tensile at the inner surface and compressive at the outer surface. σ_r becomes 0 at the inner and outer surfaces, and tensile through the wall.

When $T_i > T_o$, the opposite happens. The magnitude of the stress becomes maximum for σ_θ at the inner surface.

When assuming that the through-wall temperature distribution is linear, the following expression is obtained.

$$\sigma_\theta = \sigma_z = \pm \frac{\alpha E}{2(1 - \nu)}(T_i - T_o) \quad (6.8)$$

where “+” is for the stress at the outer surface, and “−” for that at the inner surface. Equation (6.8) is often sufficient as an approximated estimation of the thermal stress due to a through-thickness temperature gradient.

[2] Thermal stresses near the liquid level of a cylindrical vessel

The solution calculated for thermal stresses near the liquid level of a cylindrical vessel is useful for making simple estimations of thermal stresses such as the one shown in (c) or (d) in Fig. 6.4. When a sufficiently long thin-walled cylinder is subjected to a linear temperature gradient $\Delta T/\Delta l$ in its axial direction (Fig. 6.8), both circumferential stress and axial bending stress occur since different circumferential thermal expansions along the axial direction. The elastic thermal stress distribution in this case is shown on the right side of the figure. The maximum stress occurs at the break point of the temperature profile (cross section A), and the stress value is expressed by the following equations. The process of deriving these equations from the cylindrical shell theory is described in Ref. [4].

$$\sigma_{\theta} = \frac{1}{4} \left\{ -\sqrt{\frac{(1-\nu^2)}{3}} \pm \nu \right\} \times \left\{ \sqrt[4]{\frac{3}{(1-\nu^2)^3}} \right\} \sqrt{Rt} E \alpha \frac{\Delta T}{\Delta l} \quad (6.9)$$

$$\sigma_z = \pm \frac{1}{4} \left\{ \sqrt[4]{\frac{3}{(1-\nu^2)^3}} \right\} \sqrt{Rt} E \alpha \frac{\Delta T}{\Delta l} \quad (6.10)$$

where “+” corresponds to the stress at the outer surface, and “−” the stress at the inner surface. Equations (6.9) and (6.10) indicate that the near-liquid-surface thermal stress increases proportionally with the square root of the product of the radius R and the wall thickness t , and along with the axial temperature gradient.

When a temperature profile has two breakpoints (A and B) distant by Δl (right side of Fig. 6.8), the stress distribution becomes asymmetric with peaks at cross sections A and B (with opposite signs). This calculation gives an approximate solution for the thermal stress caused by thermal stratification, shown in Fig. 6.4 (d). This stress distribution is obtained by superposing the stress distribution which has a peak at cross section A with the stress distribution which has the same shape and peak with the opposite sign at cross section B (right side of Fig. 6.8).

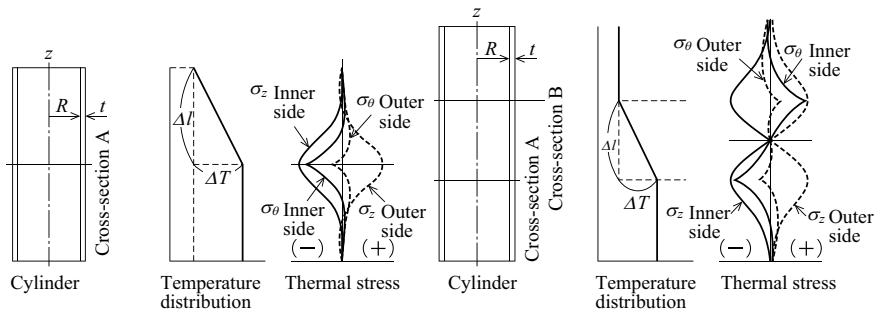


Fig. 6.8 Thermal stress due to axial temperature gradient in a cylinder

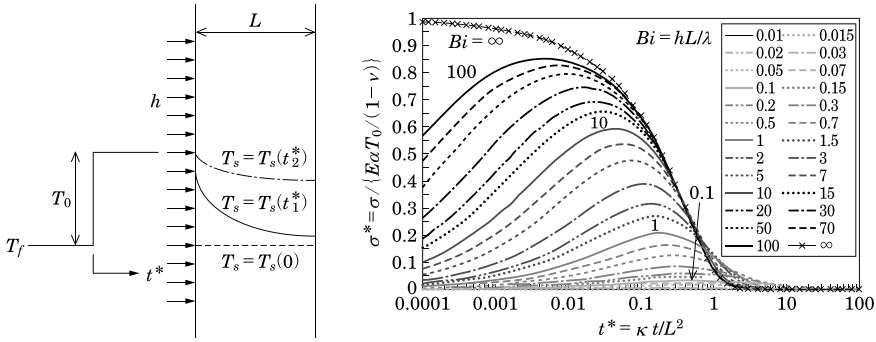


Fig. 6.9 Thermal stress response on a flat plate surface to a step change in fluid temperature

[3] Thermal stress response to a step change in fluid temperature in a flat plate

The thermal stress response to a step change of fluid temperature on a flat plate can provide a simplified estimation of the transient response characteristics of a thermal stress shown in Fig. 6.4f. The solutions for the thermal stress response to a step change T_0 in fluid temperature T_f in a flat plate with a thickness of L , shown in Fig. 6.9, are derived from the one-dimensional non-steady heat conduction theory and the thermoelastic theory. However, since these solutions are mathematically expressed by the sum of a dimensionless series, it is necessary to perform calculation every time when they are applied in design. Therefore, efforts have been made to compile the solutions in a chart for practical use. A famous one is the Heisler chart [5], in which the temperature response of the structure to a step change in fluid temperature are expressed by the Fourier number $t^* = \kappa t / L^2$ (dimensionless time, κ : thermal diffusivity of the structure, t : time) and Biot number: $Bi = hL / \lambda$ (dimensionless heat transfer coefficient, h : heat transfer coefficient, λ : thermal conductivity of the structure).

The following response chart, providing thermal stress σ to a step change in fluid temperature in a flat plate, was extended from the Heisler chart. The dimensionless thermal stress on a heat transfer surface in response to a step change in fluid temperature $\sigma^* = \sigma / \{E \alpha T_0 / (1 - \nu)\}$, can be described as a function of dimensionless time t^* by taking Biot number Bi as a parameter, as shown in Fig. 6.9.

Since knowledge on heat transfer engineering is required to solve the above-noted non-steady thermal stress problems, a basic textbook [6] and a practical guide book [7] are provided in the reference list.

6.2.3 Thermal Stress Analysis Using the Finite Element Method

Thermal loads which are important for the fast reactor often cause non-steady thermal transient stresses. The estimation of these stresses, in most cases, requires thermal stress analysis using the finite element method (FEM). In the following paragraphs, the theoretical background of solving methods will be described. Examples of thermal transient stress analysis using commercial FEM codes are introduced in Chap. 4.

[1] Non-steady heat conduction analysis

Thermal stress analysis using the finite element method consists of two steps: temperature analysis (non-steady heat conduction analysis), and thermal stress analysis using the result of the first step as an input load. First, the solving method for non-stationary heat conduction problem using the finite element method, which was not covered in Chap. 4, is outlined as follows. The temperature response of reactor components is often calculated using the external input of the fluid temperature variations, obtained from thermal hydraulic analyses, and the heat transfer coefficient. The non-stationary heat conduction problem involving heat transfer generally appears as illustrated in Fig. 6.10, and is formulated by the second-order partial differential equation with the boundary conditions described as follows (e.g., Ref. [6]).

$$\rho C \frac{\partial T}{\partial t} = \lambda \left(\frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial y^2} + \frac{\partial^2 T}{\partial z^2} \right) + Q \quad (6.11)$$

$$T = T_p \text{ on } S_1, \quad q = q_0 \text{ on } S_2, \quad q = h(T - T_f) \text{ on } S_3 \quad (6.12)$$

where: T is the temperature, t is the time, ρ is the density, C is the specific heat, λ is the thermal conductivity, Q is the heat generation per unit time, q is the heat flux, h is the heat transfer coefficient, and T_f is the external temperature.

In Eq. (6.11), the left-hand side represents the rate of temperature variation with time, and the first term of the right-hand side corresponds to the degree of spatial deviation of the temperature distribution (i.e., degree of the difference between the temperature at a point and that at an adjacent point). Namely, when the temperature

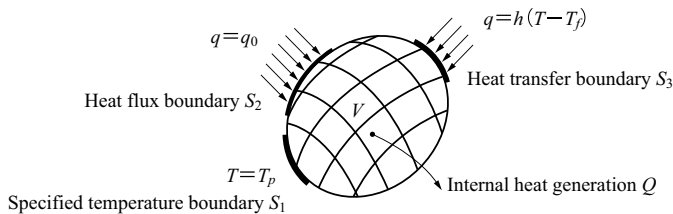


Fig. 6.10 Non-steady heat conduction problem

at a point is low compared to its surrounding area, heat flows from the surrounding area according to the temperature gradient. The rate of temperature increase on the left-hand side is determined by the amount of heat flux. The relationship between the temperature gradient and heat flux is expressed by the Fourier's law as follows.

$$q = -\lambda \frac{\partial T}{\partial x} \quad (6.13)$$

When temperature dependency of physical properties is not taken into consideration, the heat conduction problem described by Eq. (6.11) becomes a linear problem, and the analogy with the elastic problem is formed as illustrated in Table 6.3. Using this analogy, it is possible to explain the formulation of the heat conduction problem by comparison with the finite element method applied to the elastic problem described in Chap. 4.

Table 6.3 suggests that temperature T is the variable given at a nodal point in the heat conduction problem and the temperature distribution τ in an element can be expressed by the following equation using the shape function, in a similar way to Eq. (4.1) for the elastic problem.

$$\{\tau_{(k)}\} = [N_{(k)}]\{T_{(k)}\}, \text{ on } V^{(k)} \quad (6.14)$$

By differentiating Eq. (6.14), the temperature gradient is derived. Then, heat flux is calculated using the Fourier's law. Next, to satisfy both Eqs. (6.11) and (6.12), the variational method (Ritz method) is applied. It is an approach similar to the principle of minimum potential energy, described in Chap. 4. As for a non-steady heat conduction problem involving composite boundary conditions for specified temperature and heat flux, as well as internal heat generation, the accuracy of the approximation is estimated by functional $J[T]$, expressed as follows.

$$J[T] = \int_{V^{(k)}} \left\{ \frac{1}{2} \frac{\partial T}{\partial x_i} K_{ij} \frac{\partial T}{\partial x_j} + c \rho T - Q T \right\} dV + \int_{S_2} q_0 T dS + \int_{S_3} \frac{1}{2} h (T - T_f)^2 dS \quad (6.15)$$

The determination of steady-state values in Eq. (6.15), after substitution of both temperature gradient and heat flux derived from Eq. (6.14), gives a heat conduction governing equation for elements—corresponding to the element stiffness equation

Table 6.3 Analogy between elastic problem and heat conduction problem

Elastic problem	Displacement, U	Load, F	Strain, ε	Stress, σ	Hooke's law, $\sigma = E\varepsilon$
Heat conduction problem	Temperature, T	Heat generation, Q	Temperature gradient, $\delta T / \delta x$	Heat flux, q	Fourier's law, $q = -\lambda \delta T / \delta x$

for the elastic problem, Eqs. (6.4–6.5) in a matrix form as follows [8]:

$$[K_{(k)}]\{T_{(k)}\} + [C_{(k)}]\left\{\frac{\partial T_{(k)}}{\partial t}\right\} = \{F_{(k)}\} \quad (6.16)$$

where,

$$\begin{aligned} [K_{(k)}] &= \int_{V^{(k)}} [B]^T \lambda [B] dV, \quad [B] = [\partial] [N], \quad [C_{(k)}] = \int_{V^{(k)}} [N]^T \rho c [N] dV \\ \{F_{(k)}\} &= \int_{V^{(k)}} [N]^T Q dV - \int_{S_2^{(k)}} [N]^T q_0 ds - \int_{S_3^{(k)}} [N]^T h(T - T_f) ds \end{aligned}$$

The heat conduction equation for the entire system can be described by integration of the equation above for all elements and addition of boundary conditions.

$$[K]\{T\} + [C]\left\{\frac{\partial T}{\partial t}\right\} = \{F\} \quad (6.17)$$

In the previous equation, $[K]$ is called the heat conduction matrix, $[C]$ the heat capacity matrix, and $\{F\}$ the heat flux vector.

In the case of a steady heat conduction problem, since the second term of the left-hand side becomes 0, the equation becomes:

$$[K]\{T\} = \{F\} \quad (6.18)$$

On the other hand, in the case of a non-steady heat conduction problem, the governing equation, Eq. (6.17), is not discretized temporally although it is discretized spatially by means of the finite element method. Since Eq. (6.17) is a first-order ordinary differential equation regarding time, the time difference scheme based on the Crank-Nicolson method, a general method for the digitization of ordinary differential equation, is employed. The Crank-Nicolson method approximates the temperature and its temporal derivative at time, $t + \Delta t/2$, using the values at times t and $t + \Delta t$, as follows:

$$\left\{T\left(t + \frac{\Delta t}{2}\right)\right\} = \frac{\{T(t + \Delta t)\} + \{T(t)\}}{2} \quad (6.19)$$

$$\left\{\frac{\partial T}{\partial t}\left(t + \frac{\Delta t}{2}\right)\right\} = \frac{\{T(t + \Delta t)\} - \{T(t)\}}{\Delta t} \quad (6.20)$$

The substitution of Eqs. (6.19) and (6.20) into Eq. (6.17) and rearrangement gives the following equation.

$$\left(\frac{1}{2}[K] + \frac{1}{\Delta t}[C]\right)\{T(t + \Delta t)\} = \left(-\frac{1}{2}[K] + \frac{1}{\Delta t}[C]\right)\{T(t)\} + \{F\} \quad (6.21)$$

When the initial value $\{T(0)\}$ for Eq. (6.21) is given, it is possible to estimate the temporal variation of the temperature distribution by performing sequential calculations.

[2] Thermoelastic analysis

It is possible to calculate the stress response at each time step by giving the nodal temperatures at each time step, which are obtained from non-steady heat conduction analysis of the sequential stress analysis model. When the temperature $\{T\}$ is input at the nodal points of the stress analysis model, the temperature distribution in the element is obtained by the shape function. Using this temperature distribution, it is possible to obtain the thermal strain $\{\varepsilon^t\}$ in the element by multiplying the difference in temperature from the reference temperature, $\{T_R\}$, at which the thermal strain is zero, and the linear expansion coefficient α of the material.

$$\{\varepsilon^t\} = \alpha[N](\{T\} - \{T_R\}), \text{ on } V^{(k)} \quad (6.22)$$

Since the thermal strain mentioned above is considered as one of the initial strains, it is substituted into Eq. (4.3) to yield the following equation.

$$\{\sigma\} = [D](\{\varepsilon\} - \{\varepsilon^t\}) \quad (6.23)$$

When using Eq. (6.23) instead of Eq. (4.3), the load vector in Eq. (4.8) is expressed by the following equation, showing that an apparent load term resulting from the existence of the thermal strain $\{\varepsilon^t\}$ is added.

$$\{F_{(k)}\} = \int_{V^{(k)}} [N]^T \{f\} dV + \int_{S^{(k)}} [N]^T \{T\} ds + \int_{V^{(k)}} [B]^T [D] \{\varepsilon^t\} dV \quad (6.24)$$

The replacement of the load vector in Eq. (4.8) with the one in Eq. (6.24) makes it possible to perform the thermoelastic analysis in a similar way to the elastic problem described in Chap. 4.

6.3 High Temperature Behavior and Analysis

6.3.1 Simplified Inelastic Analysis Method

Since creep and elastic-plastic deformation are likely to occur in structures at high temperatures, subsequent phenomena must be considered for the evaluation of stress and strain. To estimate the elastic-plastic creep behavior of materials, two

methods are available: a method to estimate the inelastic behavior using a simplified method based on elastic analysis results, and a method to directly obtain the solution for elastic–plastic creep using the inelastic finite element method. Between these methods, the former method based on the elastic analysis is more frequently used in design. Therefore, simplified inelastic analysis methods which are required for the evaluation of various types of failure modes, are provided by design rules as part of the strength evaluation. Typical simplified inelastic analysis methods adopted by the “Fast Reactor Rule” are described in the following paragraphs.

[1] O’Donnell-Porowski method

Even when internal pressure or dead weight do not cause excessive deformations as primary stress, excessive deformation can be caused by the interaction of superposed cyclic thermal stress (secondary stress). The “ratchet deformation,” a progressive deformation due to cyclic loading, is one example, and its mechanism of occurrence has been previously described in Sect. 3.4.2 in Chap. 3 using a simple model.

The phenomenon where deformation comes from the superposition of the primary stress σ_m caused by constant loads (such as internal pressure), and the cyclic thermal stress σ_t is called “thermal ratchet.” It does not necessarily occur at high temperature. Chapter 5 explains that the Bree diagram shown in Fig. 6.11a is used in LWR design to prevent the occurrence of “thermal ratchet.” This diagram gives the limit of the combination of the load-controlled stresses (described by the parameter $x = \sigma_m/\sigma_y$) and the displacement-controlled stresses (described by the parameter $y = \sigma_t/\sigma_y$).

At high temperatures, creep may enhance progressive deformation, even when no plastic ratchet occurs. This phenomenon is called creep ratchet. The generation mechanism is estimated as follows:

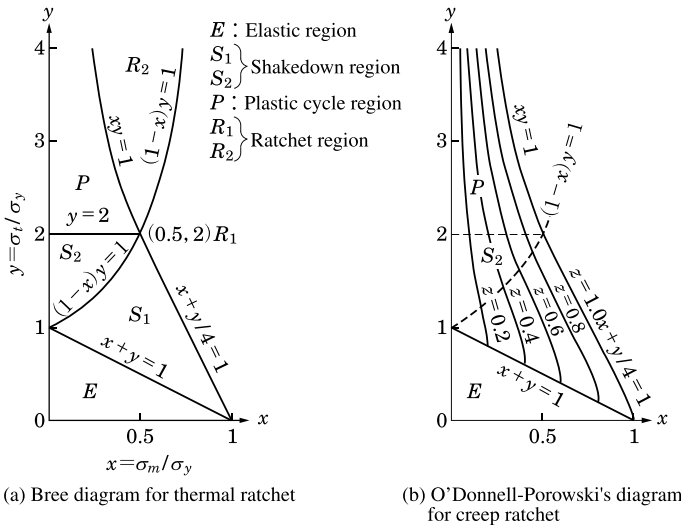


Fig. 6.11 Bree diagram and O'Donnell-Porowski's diagram

- In the model assumed in the Bree diagram, a residual stress component exists at the unloading phase, and is effective to prevent plastic ratchet (Fig. 5.12). However, since the component is relaxed by creep, a progressive deformation occurs during the reloading phase, even when the thermal stress cycle does not cause plastic ratchet.
- The stress distribution at the unloading phase (Fig. 5.12) shows that the stress is higher than the primary stress σ_m , caused by constant loads in some regions in the wall thickness. Indeed, the increase in the wall-averaged strain due to creep has been enhanced compared with the case where only the primary stress σ_m exists. This mechanism is called enhanced creep.

The O'Donnell-Porowski method is a simplified method to estimate the above-mentioned enhanced creep strain. As illustrated in Fig. 5.16, an elastic region is always present near the center of the wall thickness during the unloading phases in regimes *E*, *S*, and *P*. This region is called an elastic core, and the stress in the region is given by the following equation [9].

$$\sigma_c = z\sigma_y, \quad z = \begin{cases} 1 + y - 2\sqrt{(1-x)y} & (S_1) \\ xy & (S_2, P) \end{cases} \quad (6.25)$$

The O'Donnell-Porowski's diagram for creep ratchet shown in Fig. 6.11b is obtained by plotting z , which represents the magnitude ratio of the stress in the elastic core (core stress) to the yield stress, in the Bree diagram indicating thermal ratchet limitation (Fig. 6.11a).

The core stress is relaxed when plant components are subjected to high-temperature operation. In the O'Donnell-Porowski method, however, the creep strain is calculated by assuming that the stress determined by Eq. (6.25) is uniformly present over the wall thickness without changing its value during high temperature service, and thereby, the enhanced creep strain is estimated in a conservative manner.

[2] Neuber's rule

Neuber's rule evaluates in a simple way the strain concentration at a local structural discontinuity, as shown in Fig. 6.12. Originally, this rule was proposed to evaluate the strain concentration at the notch root of a round bar subjected to shear strain. However, it is now widely used in design since it can evaluate the elastic-plastic strain concentration in quite a simple manner, as shown in the following equation.

$$\left(\frac{\sigma}{\sigma_n}\right)\left(\frac{\varepsilon}{\varepsilon_n}\right) = K^2 \quad (6.26)$$

where σ and ε are the stress and strain at the notch root, σ_n and ε_n are stress and strain in the smooth part, and K is the elastic stress concentration factor. When the elastic solution is obtained (Fig. 6.12), it is possible to predict in a simple way the elastic-plastic solution from the intersection between the hyperbolic stress-strain

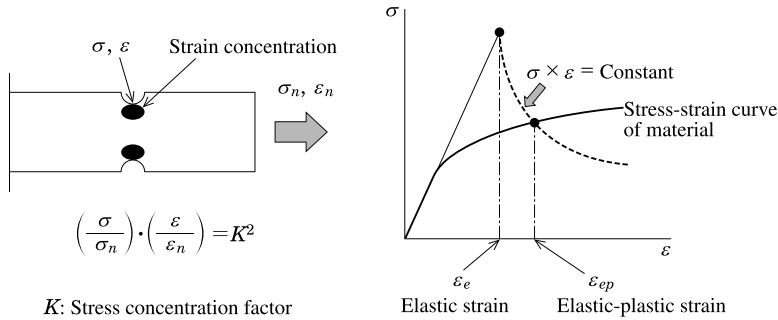


Fig. 6.12 Neuber's rule

redistribution curve according to Eq. (6.26) and the stress–strain relationship curve of the material.

[3] Analysis method using the elastic follow-up factor

A simple analysis method is introduced for the strain concentration and stress relaxation behaviors due to elastic–plastic creep using the elastic follow-up concept described in Sect. 3.3.3 in Chap. 3.

Figure 3.21 illustrates the elastic follow-up concept, when the behavior of a bar with variable cross sections becomes similar to a displacement-controlled type one. It also means that the gradient of the stress–strain curve is steeper, especially in the case when the compliance of the high stiffness portion is small compared to the low stiffness one. In the reverse case, the gradient becomes smaller since the behavior in the low stiffness portion becomes like a load-controlled type one. As described above, the gradient in the stress–strain diagram can be used as an index to characterize the behavior between a load-controlled or a displacement-controlled type. Focusing on the significance of the gradient in the stress–strain diagram, a simple analysis model using the elastic follow-up factor “ q ,” which is defined by the following equation, is provided by the design rule [1, 2].

$$q = - \frac{\Delta \varepsilon - \Delta \sigma / E}{\Delta \sigma / E} \quad (6.27)$$

where, q is the elastic follow-up factor, and $\Delta \sigma$ and $\Delta \varepsilon$ are respectively the amount of variation in stress and strain, when stress redistribution occurs due to elastic–plasticity or creep. E is Young's modulus. More specifically, $q = \infty$ under the load-controlled condition, $q = 1$ under the displacement-controlled condition, and $1 < q < \infty$ under an elastic follow-up condition. This concept is universal, and it allows the analysis of general structures similarly to the analysis of a variable cross-section bar, providing that the proper elastic follow-up factor q is obtained for each individual structure. However, the value of q for actual structures depends on their shape and the loads. Therefore, $q = 3$ has been specified by the rule [2] as a design factor

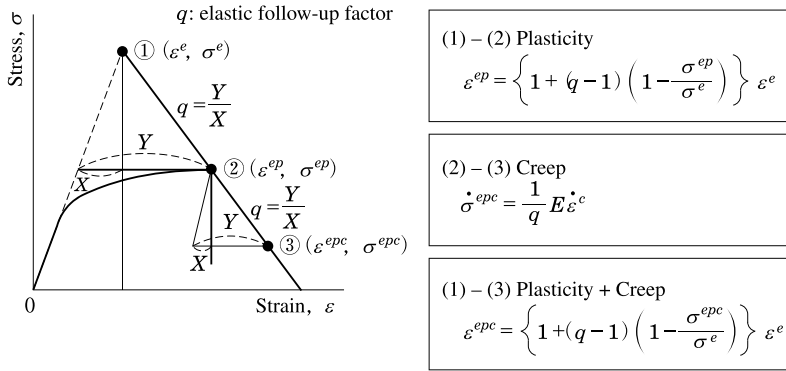


Fig. 6.13 Simplified inelastic analysis method based on the elastic follow-up factor q

that conservatively envelopes the behavior of fast reactor components toward the load-controlled side.

Figure 6.13 illustrates the method which predicts the elastic–plastic creep behavior based on the elastic analysis result using the elastic follow-up factor expressed by $q = Y/X$ in the figure.

- (1) We assume that σ^e and ε^e are respectively the stress and strain obtained from the elastic analysis.
- (2) The redistribution of stress and strain due to nonlinear behavior of the material occurs along the straight line drawn from the elastic solution point to the lower right side of elastic follow-up factor q in the stress–strain diagram. Therefore, the elastic–plastic solution is represented by the intersection between the material stress–strain curve and the straight line following q . Assuming that σ^{ep} and ε^{ep} are the stress and strain in the elastic–plastic condition, the following equation is derived from the graphical solution shown in Fig. 6.13.

$$\varepsilon^{ep} = \left\{ 1 + (q - 1) \left(1 - \frac{\sigma^{ep}}{\sigma^e} \right) \right\} \varepsilon^e \quad (6.28)$$

- (3) Next, in the case of creep deformation, it is possible to predict the stress relaxation rate in an elastic follow-up field using q , knowing that the numerator of Eq. (6.27) corresponds to the increase in creep strain and that the denominator relates to the resulting decrease in elastic strain. Therefore, since $1/q$ of the resulting creep strain contributes to stress relaxation, the stress relaxation rate becomes $1/q$ times smaller than in the case of a purely displacement-controlled condition.

$$\dot{\sigma} = -\frac{1}{q} E \dot{\varepsilon}^c \quad (6.29)$$

Using Eq. (6.29) with the stress predicted by Eq. (6.28) as the initial value, we obtain the stress relaxation history as well as the stress and strain in the elastic–plastic creep condition after relaxation, σ^{epc} and ε^{epc} .

$$\varepsilon^{\text{epc}} = \left\{ 1 + (q - 1) \left(1 - \frac{\sigma^{\text{epc}}}{\sigma^e} \right) \right\} \varepsilon^e \quad (6.30)$$

Equations (6.28)–(6.30) can be applied extendedly to the stress and strain ranges under cyclic loading without change.

6.3.2 Elastic–Plastic Creep Analysis Using the Finite Element Method

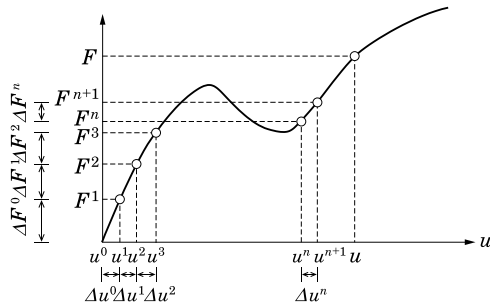
Research and development are currently conducted to allow the direct application to design of the inelastic analysis using the finite element method in consideration to the elastic–plastic creep behavior of material. Indeed, the excessively large evaluation margin associated with the use of a simplified evaluation method based on the elastic analysis impairs the pursuit of economically competitive plant design. So far, simplified evaluation methods have been mainly used for design evaluation, and the inelastic analysis using the finite element method is often employed as a confirmation of the validity and margins incorporated in simplified evaluation methods. The inelastic analysis method is more complex and it requires a higher-level of knowledge and experience, as well as more calculation time and additional costs compared to elastic analysis methods. In addition, the inelastic analysis method has the following technical problem: its solution depends on the selection of a constitutive equation and a load history, which are unknown at the design stage.

Regarding the constitutive equations, more advanced equations (e.g., Ref. [10]) have been proposed recently and more accurate experimental simulations have become possible. On the other hand, the application of these constitutive equations to design has both advantages and disadvantages, as listed in Table 6.4. Under these considerations, a design method using inelastic analysis based on the classical constitutive equation has been proposed in the LWR field. In the fast reactor field, a study is underway on the application to design of both classical constitutive equations and more elaborate constitutive equations. Yet no analysis or evaluation method has been specified from the perspective of application to design, and this remains a technical problem.

Various methods exist to perform elastic–plastic creep analyses using the finite element method (e.g., Ref. [8]). Next, a typical example is outlined. The incremental method is commonly used for analysis of nonlinear phenomena, such as plasticity and creep. It divides load or time into small steps to solve the equilibrium equations in each step by linearization, as described in Fig. 6.14. Smaller increments provide

Table 6.4 Characteristics of various constitutive equations

Constitutive equation	Classical constitutive equations (isotropic hardening rule, kinematic hardening rule, etc.)	Advanced constitutive equation (two surface model, Chaboche model, Ohno-Wang model, etc.)
Advantage	Simple and easy to understand Relatively small effect of load history on the result Integrated into a general-purpose FEM code as a standard function	Since it is capable of accurately simulating the nonlinear behavior of material, the stress and strain are predicted accurately if parameters are appropriately determined
Disadvantage	Since it is not capable of accurately simulating the hardening/softening behavior of material, the prediction of stress and strain is not accurate enough	A great deal of material data is required A certain level of skill is required to perform the analysis

Fig. 6.14 Nonlinear calculation using the incremental method

a satisfactory convergence of the solution. In addition, the incremental method can follow the response of a structure quite well along its load history.

The incremental form of Eq. (4.3), which is one of the constitutive equations for the elastic problem described in Chap. 4, is obtained by using $[D^e]$ as the elastic stress-strain matrix as follows.

$$\{\Delta\sigma\} = [D^e](\{\Delta\varepsilon\} - \{\Delta\varepsilon^0\}) \quad (6.31)$$

Let's assume plastic strain increment $\{\Delta\varepsilon^p\}$, creep strain increment $\{\Delta\varepsilon^c\}$, and thermal strain increment $\{\Delta\varepsilon^t\}$ all exist. These increments are regarded as the initial strain increment, and their substitution into Eq. (6.31) yields the following equation.

$$\Delta\{\sigma\} = [D^e](\{\Delta\varepsilon\} - \{\Delta\varepsilon^p\} - \{\Delta\varepsilon^c\} - \{\Delta\varepsilon^t\}) \quad (6.32)$$

Here, the plastic strain increment $\{\Delta\varepsilon^p\}$ is expressed by the following equation using the yield function f , which is given by Eqs. (3.8) and (3.9) as a plastic potential, and the associated flow rule given by Eq. (3.14)

$$\{\Delta \varepsilon^p\} = \Delta \lambda \left\{ \frac{\partial f}{\partial \sigma} \right\} \quad (6.33)$$

In addition, as the example case of a classical isotropic hardening rule, the yield function f is a function of stress and plastic strain, and the following equation holds in a state where plastic deformation proceeds.

$$\Delta f = \left\{ \frac{\partial f}{\partial \sigma} \right\}^T \{\Delta \sigma\} + \left\{ \frac{\partial f}{\partial \varepsilon^p} \right\}^T \{\Delta \varepsilon^p\} = 0 \quad (6.34)$$

The elimination of $\Delta \lambda$ from Eqs. (6.32) to (6.34) and the stress–strain rearrangement yield the following equation.

$$\{\Delta \sigma\} = ([D^e] + [D^p])(\{\Delta \varepsilon\} - \{\Delta \varepsilon^c\} - \{\Delta \varepsilon^t\}) \quad (6.35)$$

where, $[D^p]$ is called the plastic stress–strain matrix, and is expressed by the following equation, which includes a yield function.

$$[D^p] = - \frac{[D^e] \left\{ \frac{\partial f}{\partial \sigma} \right\} \left\{ \frac{\partial f}{\partial \sigma} \right\}^T [D^e]}{- \left\{ \frac{\partial f}{\partial \varepsilon^p} \right\}^T \left\{ \frac{\partial f}{\partial \sigma} \right\} + \left\{ \frac{\partial f}{\partial \sigma} \right\}^T [D^e] \left\{ \frac{\partial f}{\partial \sigma} \right\}} \quad (6.36)$$

When using Eq. (6.35) instead of Eq. (4.3), the element stiffness equation expressed by Eq. (4.5) is rewritten as

$$[K_{(k)}] \{\Delta U_{(k)}\} = \{\Delta F_{(k)}\} \quad (6.37)$$

$$[K_{(k)}] = \int_{V^{(k)}} [B]^T [D^e] [B] dV + \int_{V^{(k)}} [B]^T [D^p] [B] dV \quad (6.38)$$

Since the stiffness matrix in Eq. (6.37) includes the yield function as understood from Eqs. (6.36) and (6.38), it depends on the plastic strain and stress before increment. Since the stress–strain state before increment is specified by displacement, the stiffness matrix in Eq. (6.37) can be expressed also as a function of displacement vector $\{U_{(k)}\}$.

In addition, the load vector increment in Eq. (6.37) becomes a function of the increments of internal state variables such as creep strain and thermal strain increments, as expressed below.

$$\begin{aligned}
\{\Delta F_{(k)}\} = & \int_{V^{(k)}} [N]^T \{\Delta f\} dV + \int_{S^{(k)}} [N]^T \{\Delta T\} dS + \int_{V^{(k)}} [B]^T [D^e] \{\Delta \varepsilon^e\} dV \\
& + \int_{V^{(k)}} [B]^T [D^p] \{\Delta \varepsilon^e\} dV + \int_{V^{(k)}} [B]^T [D^e] \{\Delta \varepsilon^t\} dV + \int_{V^{(k)}} [B]^T [D^p] \{\Delta \varepsilon^t\} dV
\end{aligned} \quad (6.39)$$

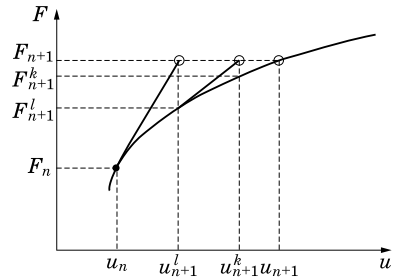
Therefore, the overall stiffness equation, which is the integration of Eq. (6.37) over all elements, also becomes a nonlinear equation in which the stiffness matrix depends on the displacement vector $\{U\}$, expressed in the following equation. Accordingly, this equation is solved by the incremental method in each step via linearization.

$$[K(U)] \{\Delta U\} = \{\Delta F\} \quad (6.40)$$

Specifically, assuming a great number of load steps $\{F_0\} \dots \{F_n\} \{F_{n+1}\} \dots$, and assuming that $\{U_n\}$ is obtained, $\{U_{n+1}\}$ at the next time step is sequentially calculated. One of such calculation methods is the Newton–Raphson method. From Fig. 6.15, you can understand that $\{U_{n+1}^l\}$ calculated from $\{U_n\}$ according to the following equation is the approximate solution for $\{U_{n+1}\}$. When this calculation is repeated, the solution gradually approaches $\{U_{n+1}\}$.

$$[K(U_n)]\{U_n\} + \frac{d[K(U_n)]}{d\{U_n\}}(\{U_{n+1}^l\} - \{U_n\}) = \{F_{n+1}\} \quad (6.41)$$

Fig. 6.15 Newton–Raphson method



6.4 “Design by Analysis” of High-Temperature Components

6.4.1 *Overview of the Design Codes for High-Temperature Structures*

The “design by analysis” approach was proposed in Section III of the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Design Code (hereafter, “Sec. III”) and has become the basic concept of structural design of the nuclear components. It significantly improves freedom in design by requiring “detailed design in consideration of the failure mode.” Hence, it enables the design of components with new structures as well as lightweight components [11]. This design rule incorporates the concept of plastic design on the premise of application to ductile materials. A feature of this rule is the prevention of failure related to plastic strain by limiting the potential stress obtained from the elastic analysis. This is performed since various insights concerning the local plastic strain behavior of a structure in a global elastic state.

On the other hand, in fast reactor designs, additional provisions to Sec. III were required, due the time-dependent failure modes, as well as the inelastic behavior originated by the decrease in the yield stress of material and creep deformation resulting from high-temperature operation affects the failure modes. Proper evaluation of these phenomena is crucial.

As an example of the method of designing high-temperature nuclear components, Table 6.5 displays the relationship between the rules and failure modes considered in the Japanese “Rules on Fast Reactors” [2]. Foreign design codes for high-temperature structures [9, 12] also have similar frameworks.

In Table 6.5, concerning the “strain limits,” “limits for creep-fatigue damage” and “prevention of buckling,” specific analysis methods are respectively provided in the rule to apply the design criteria related to the inelastic behavior of a structure. These methods were extended from the simplified analysis methods that predict the inelastic behavior of a structure leading to failure. They use the potential stresses obtained from the elastic analysis, which is the feature of Sec. III. Since these simplified methods are specified to give a conservative solution, they might introduce additional margins in addition to the necessary design margins that are ensured by the design factors. Therefore, regarding the dominant load in the fast reactor is the thermal transient stress, these methods are designed to improve the evaluation accuracy when the long-term primary stress is significantly lower than the secondary stress. In addition, a detailed analysis method that directly obtains the stress and strain using the inelastic finite element analysis method is permitted. Among these methods, an overview of simplified methods is provided, to analyze the stress-strain state and the corresponding limiting values, with a focus on failure modes that are unique to the fast reactor.

Table 6.5 Failure modes and rules that are taken into account in the “Rules on Design and Construction for Nuclear Power Plants, Part II, Rules on Fast Reactors”

Rule	Elastic analysis		Elastic analysis or inelastic analysis		
Failure mode	Primary stress limits	Special stress limits	Strain limits	Limits for Creep-fatigue damage	Prevention of buckling
Ductile fracture, plastic collapse	○	Δ	Δ		
Excessive plastic deformation	○	Δ	○		
Fatigue failure			Δ	○	
Elastic–plastic buckling	◇		Δ		○
Creep rupture	○	Δ	Δ		
Excessive creep deformation	○	Δ	○		
Creep-fatigue failure			Δ	○	
Creep buckling	Δ		Δ		○

○: Main rules, Δ: Supplementary rules, ◇: Prevention of collapse

6.4.2 Primary Stress Limits

S_t is the primary stress limit in fast reactors, aiming at preventing creep rupture, which is a failure mode unique to high-temperature structures. This primary stress limit is considered on top of the low-temperature limit S_m .

S_t is a time-dependent limit, which is determined by the minimum of the values listed below:

- (1) 2/3 of the minimum stress to cause creep rupture at temperature T and at time t .
- (2) 0.8 times the minimum stress to cause the tertiary creep at temperature T and at time t .
- (3) The minimum stress to cause the total strain of 1% at temperature T and at time t .

To develop the above-listed limits, it is necessary to formulate the time to rupture t_R as a function of T and σ and quantitatively describe the change in creep strain ε^c using T , σ , and t . For this purpose, the type of characteristic equation has to be specified. Then, the characteristic equation itself is determined by statistical analysis of suitable test data, which evaluates with reliability the variation of materials properties and the influence of most parameters.

Concerning the creep rupture time t_R , the following type of characteristic equation is assumed using the Larson-Miller parameter described in Sect. 3.2.2 in Chap. 3.

$$T(\log t_R + C) = A_0 + A_1 \log \sigma + A_2(\log \sigma)^2 \quad (6.42)$$

where C , A_0 , A_1 , and A_2 are constants and can be determined by regression analysis of the creep rupture data as shown in Fig. 3.17.

In the fast reactor design rule, a Blackburn-type's equation is used for the characteristic equation of creep strain ε^c , as follows.

$$\varepsilon^c = C_1(1 - e^{-r_1 t}) + C_2(1 - e^{-r_2 t}) + \dot{\varepsilon}_m t \quad (6.43)$$

C_1 , C_2 , r_1 , and r_2 in Eq. (6.43) are functions of creep rupture time t_R . $\dot{\varepsilon}_m$ is the secondary creep rate, which is formulated by the following equation, based on the Monkman–Grant relationship.

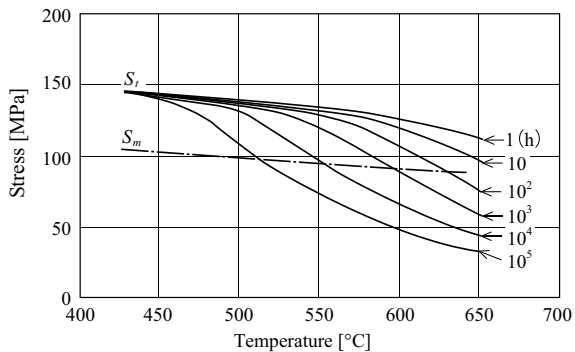
$$\dot{\varepsilon}_m = F_{\exp} \left(-\frac{Q}{RT} \right) t_R^{-\lambda} \quad (6.44)$$

The creep strain Eq. (6.43) is formulated as a function of creep rupture time t_R as the creep strain and creep rupture time are correlated. Indeed, creep strain, in general, is unlikely to occur in the case of a harder material that exhibits a longer creep rupture time.

The comparison between S_m and S_t values for 304SS, shown in Fig. 6.16, suggests that S_t becomes increasingly smaller than S_m as temperature becomes higher and as the time period becomes longer. This tendency determines the limiting value.

Loads are classified into long-term and short-term ones, depending on whether the creep effect should be taken into account or not. For instance, the general primary membrane stress resulting from long-term load is designated as P_m , and the general primary membrane stress resulting from short-term load is designated as P_m^* . Here, P_m^* is a stress that acts only for a short period of time, such as seismic load and impact pressure, and is caused by a load that does not cause creep deformation. The

Fig. 6.16 Comparison between S_m and S_t values for 304SS



limit for service conditions A and B is expressed as follows in consideration of both S_m and S_t , which are respectively the limit for low-temperature designs and the limit unique to high-temperature structures, which aim at preventing creep rupture.

$$\langle P_m + P_m^* \rangle \leq S_m \quad (6.45)$$

$$\langle P_m \rangle \leq S_t \quad (6.46)$$

6.4.3 Strain Limits

In the structural design for LWRs, gross deformation is prevented by limiting the primary plus secondary stress to twice the yield stress in principle (i.e., shakedown limit) to lead to elastic deformation, or within the regions, E , S , or P in the Bree diagram (prevention of ratchet). On the other hand, the decrease of yield stress and creep effect at high temperatures makes it difficult to create design that permits no ratchet deformation at all. Therefore, the “Fast Reactor Rule” has a provision that limits the strain within the following ranges that are considered unlikely to affect the integrity:

Average through-wall strain: 1%

Surface strain determined from the equivalent linear strain distribution through the wall thickness: 2%

Therefore, the “Fast Reactor Rule” provides an evaluation method to predict the magnitude of strain, in addition to the prevention of gross deformation which corresponds to the shakedown limit specified in the LWR Rule. The total sum of the strain magnitude predicted for some assumed deformation mechanisms is required to be within the above-noted limits.

[1] Shakedown limit in consideration of creep

Since the residual stress component which effectively prevents plastic ratchet is relaxed in the creep temperature range, shakedown may not occur even when the primary plus secondary stress is within twice the yield stress. Therefore, the primary plus secondary stress is separated into $P + Q$, which acts for a long time, and $P^* + Q^*$, which acts only for a short time and hence does not cause creep deformation. Concerning the limits for service conditions A, B, and C, the following equations are required to be satisfied by taking into account both $3S_m$ used for low-temperature design and $\overline{3S_m}$ which takes the relaxation of residual stress due to creep into consideration.

$$\langle P + P^* + Q + Q^* \rangle \leq 3S_m \quad (6.47)$$

$$\langle P + Q \rangle \leq \overline{3S_m} \quad (6.48)$$

[2] Evaluation method of creep ratchet

The “Fast Reactor Rule” is designed to allow conservative evaluation of creep ratchet including the residual stress relaxation effect, by evaluating the above-mentioned enhanced creep in a conservative manner, using the O’Donnell-Porowski method. The core stress is actually relaxed when plant components are subjected to high-temperature operation. In design, however, creep strain is calculated by assuming a constant stress $\sigma_c = 1.25\sigma_y$. This stress is obtained by multiplying Eq. (6.25) by the design factor, which is exerted uniformly over the wall thickness during high-temperature service. Therefore, the amount of creep ratchet strain is estimated in a conservative manner.

[3] Other evaluation methods of ratchet

Concerning the “strain limits,” a great deal of efforts has been put into developing methods to evaluate the progressive deformation under cyclic loading condition. This is due to the difficulty of predicting ratchet deformation. The O’Donnell-Porowski method, the most famous method to evaluate ratchet deformation, models the behavior of a cylinder subjected to combined application of constant internal pressure and a cyclic thermal bending stress caused by a through-wall temperature gradient, as shown in Fig. 6.17a. This method is incorporated into the design rules of various countries. On the other hand, the movement of the liquid surface as shown in Fig. 6.17b is anticipated in the fast reactor vessel. In this case, ratchet deformation may occur solely due to thermal stress. Since the reactor wall is exposed to high and low temperatures, respectively in the lower and upper parts, separated by the liquid surface, circumferential tensile thermal membrane stress acts in the upper part and circumferential compressive thermal membrane stress acts in the lower part to maintain the shape continuity of the structure. Since the liquid surface rises during a reactor startup due to thermal expansion, these stresses repeatedly occur at every startup process, when the wall region is affected by the liquid surface passing by. The equation to evaluate the ratchet deformation that addresses the above deformation mode [13] was proposed. The development of analysis method for every deformation mode to address all conditions, as described above, makes the overall evaluation procedure complicated. On the other hand, one approach exists to evaluate all deformation modes in a unified manner. In France, a method has been developed to determine whether ratchet deformation occurs or not using a diagram obtained from experiments [12]. In the U.K., a method to limit the strain by paying attention to the elastic region remaining in a through-wall cross section (elastic core) [14] has been adopted. The adoption of a similar method to Japan’s LWR rule is under discussion. These methods are simple, easy-to-use, and can provide good perspectives. However, they tend to be conservative because they cannot take into account the deformation resistance of every ratchet mode. Conversely, when efforts are made to create a more elaborate model to eliminate an excessive safety margin, simplicity tends to be lost.

When using an inelastic finite element analysis method in design, the constitutive equation and a method to set the load history must be appropriately selected

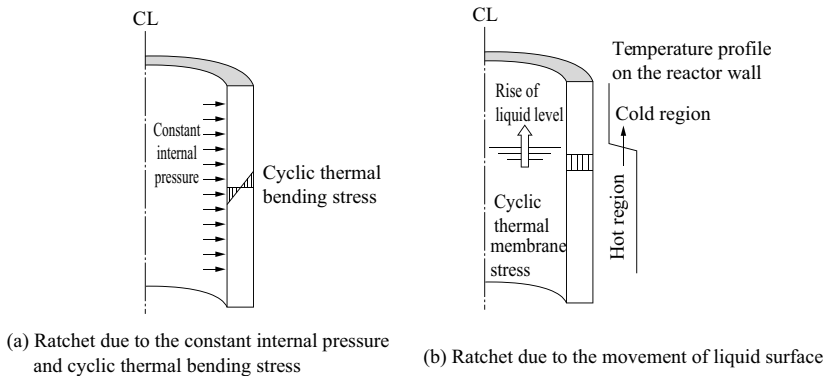


Fig. 6.17 Ratchet deformation mode assumed for fast reactors

by giving consideration to the conservativeness. Adequate consideration must be also given to the problem of computing time. Additional problems, including analysis proficiency and non-guaranty of uniqueness of the solution, remain. Therefore, inelastic finite element analysis method has not been established as a general design tool yet. However, a complicated version of a simple analysis method also poses similar problems.

The use of the inelastic finite element analysis method as a general design tool causes difficulty in setting the load history and selecting the constitutive equation, however, efforts to apply the method to design are in progress. Some of the issues encountered could be moderated by limiting the applicable scope.

6.4.4 Limits for Creep-Fatigue Damage

The lifetime (number of load cycle) of a material under a temperature condition where creep is negligible is mainly determined by the magnitude of strain (strain range) which is repeatedly applied. However, at high temperatures, the lifetime gradually decreases with the increase in holding time. Since the decrease in lifetime is assumed to be caused by the superposing effects of creep damage and fatigue damage caused by the stress applied during the holding time, this type of damage is called “creep-fatigue failure.” As the thermal stress, which is the main stress factor for the fast reactor components, is repeatedly caused by the operation cycle, such as startup and shutdown, careful attention is paid to prevent the creep-fatigue failure caused by cyclic thermal stress in the structural design.

[1] Failure criteria

The prevention of creep-fatigue failure is achieved according to the linear cumulative damage rule which limits the sum of the following values: (i) fatigue damage factor D_f

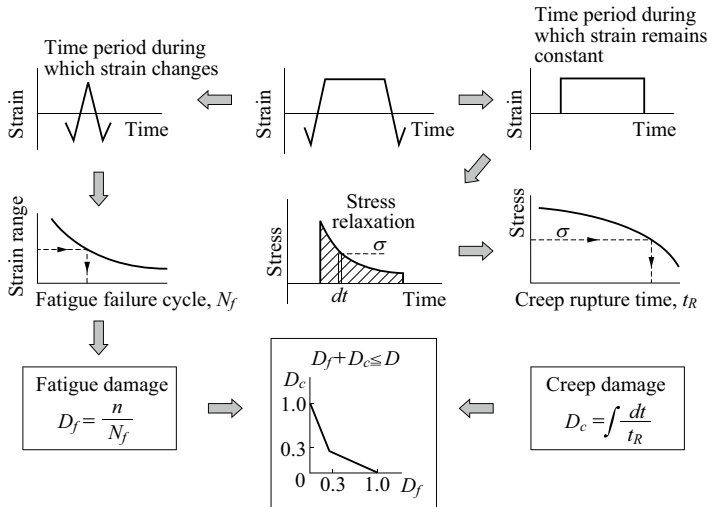


Fig. 6.18 Creep-fatigue damage estimation method

and (ii) creep damage factor D_c . Both values are separately calculated, as illustrated in Fig. 6.18, by dividing a trapezoidal strain waveform into two parts: namely, D_f for the time period during which the strain changes (corresponding to the plant startup and shutdown) and D_c for the time period during which the strain remains constant (corresponding to the steady-state plant operation). The following equation is provided.

$$D_f + D_c \leq D \quad (6.49)$$

Here, the limiting value D equals 1 when pure fatigue damage or creep damage solely occurs, while D should be less than 1 when both types of damage simultaneously occur due to their interaction.

The usage factor D_f is obtained according to Miner's rule, which linearly sums up n_i/N_{fi} . The damage caused when the strain range $\Delta\epsilon_i$ is repeatedly applied n_i times is designated as ϕ_{fi} . Then, ϕ_{fi} is represented by n_i/N_{fi} , which is the ratio of n_i to the fatigue life N_{fi} with regard to $\Delta\epsilon_i$.

$$D_f = \sum_i \frac{n_i}{N_{fi}} \quad (6.50)$$

The creep damage factor D_c is obtained by a time consumption-type method that linearly sums up t_j/t_{Rj} . The damage caused when the stress σ_j is applied for the time period t_j is designated as ϕ_{cj} . Then, ϕ_{cj} is represented by t_j/t_{Rj} , which is the ratio of t_j to the creep rupture time t_{Rj} with regard to σ_j .

$$D_c = \sum_j \frac{t_j}{t_{Rj}} \quad (6.51)$$

As described in Sect. 5.6.3 of Chap. 5, the design factor of 2 for the strain range and of 20 for the number of load cycle are incorporated into the fatigue curve used for the calculation of D_f to take into account the variation in material and the uncertainty of the stress calculation.

Concerning D_c , a design creep rupture curve reduced to 1/10 of the average creep rupture curve of a material (with regard to time) is used, and the damage value is doubled to provide an additional safety margin.

[2] Method to predict inelastic behavior

Since the above-noted failure criteria for creep-fatigue requires the strain range and the stress history as an input, it is necessary to appropriately predict the stress-strain behavior during reactor operation. Figure 6.19 schematically shows the stress-strain behavior of a structure assumed when estimating creep-fatigue lifetime. Taking a Y-shaped joint structure consisting of a member cooled by a coolant and another non-cooled member as an example, we pay attention to the stress and strain behavior at the stress concentration zone when the coolant temperature is repeatedly changed due to startup and shutdown of a plant. When the coolant temperature increases, thermal stress occurs at the concerned zone, but then the stress is relaxed due to creep during the hold time at high temperatures. Since elastic follow-up occurs at the stress concentration zone, the stress relaxation rate becomes slower compared with the condition purely under secondary stress. As for the strain, an elastic strain is caused by thermal stress, and the strain is increased at the stress concentration zone due to the concentration of elastic-plastic and creep strains. Therefore, a method is provided in the design rule in relation with the strength evaluation method to estimate in a simple way the strain range and stress history that takes into account the elastic-plastic creep deformation based on the elastic analysis result.

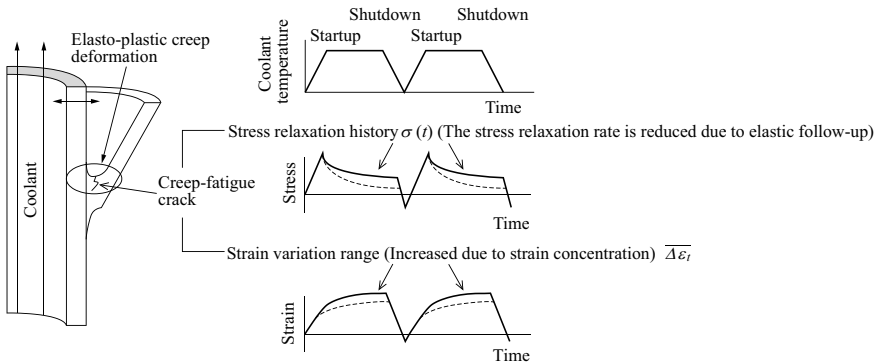


Fig. 6.19 Stress-strain behavior of structure assumed for creep-fatigue damage estimation

As for the strain range required for the calculation of the usage factor D_f , a hypothetical strain range $\overline{\Delta\varepsilon^e}$ calculated from an elastic analysis is multiplied by the strain concentration factor to estimate the strain range $\overline{\Delta\varepsilon^e}$ which considers the inelastic behavior.

In this step, since it is necessary to calculate the range of strain $\overline{\Delta\varepsilon^e}$ which is equivalent to a uniaxial strain based on the history of change of tensor components of stress and strain obtained by three-dimensional elastic analysis, one example of such calculation is described below with reference to the method specified in ASME [9]. Since the equivalent stress and equivalent strain do not have the concept of sign, it is not possible to calculate the differences after conversion to equivalent stress or strain. Therefore, it is necessary to obtain the equivalent values after calculating the variation range of each component. As for the strain, the six components of strain history, $\varepsilon_x(t)$, $\varepsilon_y(t)$, $\varepsilon_z(t)$, $\gamma_{xy}(t)$, $\gamma_{yz}(t)$, and $\gamma_{zx}(t)$, are obtained by a three-dimensional analysis.

Taking the time at which an extreme value is obtained in the history as 0, the history of the range of each component from the extreme value is given by

$$\begin{aligned}\Delta\varepsilon_x(t) &= \varepsilon_x(t) - \varepsilon_x(0) \\ \Delta\varepsilon_y(t) &= \varepsilon_y(t) - \varepsilon_y(0) \\ &\dots\end{aligned}\tag{6.52}$$

From the above-noted variation range of each component, the von Mises type equivalent strain range is calculated by

$$\begin{aligned}\overline{\varepsilon}(t) &= \frac{\sqrt{2}}{2(1+\bar{\nu})} \sqrt{(\Delta\varepsilon_x(t) - \Delta\varepsilon_y(t))^2 + (\Delta\varepsilon_y(t) - \Delta\varepsilon_z(t))^2 + (\Delta\varepsilon_z(t) - \Delta\varepsilon_x(t))^2} \\ &\quad + \frac{3}{2} (\Delta\gamma_{xy}(t)^2 + \Delta\gamma_{yz}(t)^2 + \Delta\gamma_{zx}(t)^2)\end{aligned}\tag{6.53}$$

Here, $\bar{\nu}$ is called modified Poisson's ratio. As for the perfect plastic strain, $\bar{\nu}$ equals 0.5, and Eq. (6.53) agrees with Eq. (3.11), which expresses the equivalent plastic strain. As for the elastic strain, $\bar{\nu}$ equals 0.3, while in the case of the elastic-plastic strain, $\bar{\nu}$ takes a value between the above two values. The maximum range of the equivalent strain obtained by Eq. (6.53) is determined as the strain range $\overline{\Delta\varepsilon^e}$.

Next, as methods to predict the inelastic strain concentration, two types of strain concentration factors are provided in the design rule. One is a factor based on Neuber's rule described in Sect. 6.3.1, and is designed to evaluate local strain concentration, such as in notch roots. If a tension stress S^* acts on a smooth part, the strain of the smooth part ε_n is obtained from the stress-strain curve of the material. Assuming that the stress concentration factor at the notched region is K and the stress for $K\varepsilon_n$ is $\bar{S}$,

$$\sigma/\sigma_n < K, \quad \varepsilon/\varepsilon_n > K\tag{6.54}$$

it is possible to determine the upper limit of the solution for Neuber's rule, given by Eq. (6.26), by using Eq. (6.55) according to the relationship expressed by Eq. (6.54).

$$\varepsilon = \left(\frac{\sigma_n}{\sigma}\right) K^2 \varepsilon_n \leq \left(\frac{S^*}{\bar{S}}\right) K^2 \varepsilon_n \quad (6.55)$$

Therefore, the strain range at the stress concentration zone can be predicted in a conservative manner (i.e., taking a larger value) by the following equation when assuming that the strain range of the smooth part obtained by elastic analysis is $\overline{\Delta\varepsilon^e}$.

$$\overline{\Delta\varepsilon_t} = \left(\frac{S^*}{\bar{S}}\right) K^2 \overline{\Delta\varepsilon^e} \quad (6.56)$$

The other one is a factor which aims at evaluating the concentration resulting from the overall strain redistribution in a whole structure. It provides a method using q , elastic follow-up factor, described in Sect. 6.3.1. In Eq. (6.30), by setting the upper limit of the variation range of stress σ^{epc} at $\overline{3Sm}$ in consideration of the shakedown limit value described in Sect. 6.4.3, the actual strain range can be predicted as follows in a conservative manner (i.e., taking a larger value) from the range of strain at a structure surface $\overline{\Delta\varepsilon^e}$ obtained by elastic analysis.

$$\overline{\Delta\varepsilon_t} = \left\{ 1 + (q - 1) \left(1 - \frac{\overline{3S_m}}{E \overline{\Delta\varepsilon^e}} \right) \right\} \overline{\Delta\varepsilon^e} \quad (6.57)$$

In the following paragraph, the calculation of the stress history required for evaluation of creep damage is discussed. The initial stress σ_i at the start of retention at high temperatures can be predicted by inputting the strain $\overline{\Delta\varepsilon_t}$ predicted using Eqs. (6.56) and (6.57) into the stress range-strain range relation $\Delta\sigma = \Delta\sigma(\Delta\varepsilon)$ (obtained from the cyclic stress-strain curve of a material). Then, considering that the dominant load in the fast reactor is the thermal stress, it is expected that the stress is relaxed during retention at high temperatures. However, considering that even thermal stress does not become purely displacement-controlled and that the elastic follow-up occurs to a certain degree in an actual reactor as described in Sect. 3.3.3 in Chap. 3, the stress relaxation rate $\dot{\sigma}$ is conservatively predicted from Eq. (6.29) and the upper limit of q . Using the initial stress σ_i and the stress relaxation rate $\dot{\sigma}$, the stress history $\sigma(t)$ required for calculation of the creep damage factor D_c is calculated.

[Supplement: time consumption model and ductility exhaustion model]

As for the creep damage factor D_c , the time consumption type factor t_f/t_{Rj} is simple and practically used for design. However, the direct relationship with the failure mechanism is not covered by it. For this reason, some design codes have adopted the ductility exhaustion model on the basis of the strain being related to the formation of cavity at a grain boundary, which is a creep damage mechanism described in Sect. 3.2.2 in Chap. 3. The ductility exhaustion model expresses the creep damage

using the ratio of a resulting creep strain ε_{cj} to creep rupture ductility δ_{cj} , $\varepsilon_{cj}/\delta_{cj}$. Here, assuming that the secondary creep rate $\dot{\varepsilon}_2$ solely contributes to creep damage, the following equations hold.

$$\delta_{cj} = \dot{\varepsilon}_2 t_{Rj}, \varepsilon_{cj} = \dot{\varepsilon}_2 t_j \quad (6.58)$$

Then, the ductility exhaustion $\varepsilon_{cj}/\delta_{cj}$ agrees with the time consumption t_j/t_{Rj} .

6.4.5 Prevention of Buckling

Since fast reactors are operated at higher temperatures than LWRs, creep buckling needs to be taken into account as an additional failure mode. Furthermore, as a thin-walled structure is employed, a greater attention must be given to elastic and elastic–plastic buckling compared with LWRs.

[1] Creep buckling

In the case of a high temperature structure within the creep range, even when a load smaller than the critical load for elastic and elastic–plastic buckling (time independent) is applied, creep deformation progresses with the time, and an instable phenomenon may suddenly take place at some point. This phenomenon is called creep buckling, or time-dependent buckling. As an example, we assume that a spherical aluminum shell, to the top of which a boss is attached, is compressed by a load P . Even when the load does not exceed the critical load for static buckling, when it continuously acts at a high temperature (200 °C) at which creep is still present, deformation gradually progresses, and the deformation rate may rapidly increase after a certain period of time, leading to buckling. Such time period is called critical time. As shown in Fig. 6.20, the critical time significantly changes with the level of load P . This is due to the strong nonlinearity between stress and creep strain rate, as described in Sect. 3.1.3 in Chap. 3.

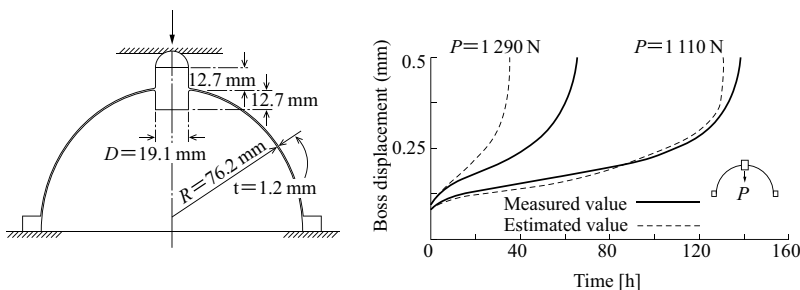


Fig. 6.20 Creep buckling of a spherical shell

[2] Buckling of a cylindrical shell due to axial compression and bending

The design rule has a provision against the time-independent buckling of a cylindrical shell due to axial compression and bending, which is different from the general provision for buckling. It specifies that the buckling of a long uniform cylindrical shell subjected to axial compression and bending is basically completely converted to axial compression buckling alone, which is evaluated by the following equation.

$$\sigma_c + \frac{\sigma_b}{y} \leq \frac{\sigma_{cr}}{f_B} \quad (6.59)$$

where

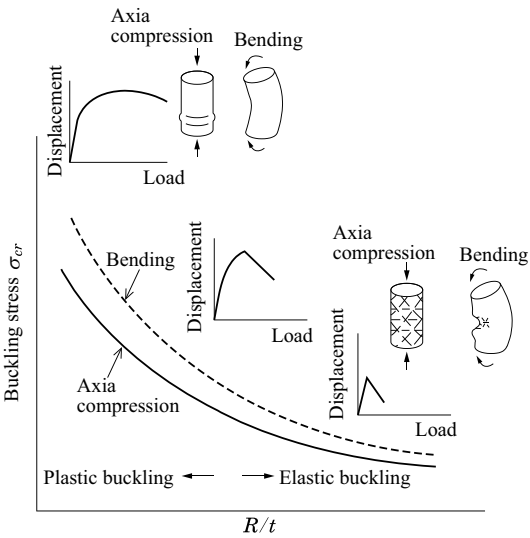
- σ_c axial compressive stress,
- σ_b maximum stress occurring in a cross-sectional surface skin due to bending moment, on the basis of elastic analysis (bending moment divided by section modulus),
- y ratio of bending buckling strength to axial compressive buckling strength,
- σ_{cr} axial compressive buckling stress of a cylindrical shell,
- f_B design factor.

As for the buckling of a cylindrical shell, since different deformation modes are likely to occur depending on the ratio of radius R to wall thickness t , R/t , in addition to the type of load, such as axial compression, bending and external pressure, it is necessary to provide an evaluation method for each mode based on the confirmation of the dominant mode by experiment, etc.

In the case of a cylinder subjected to axial compression, it is known that the buckling deformation mode (buckling waveform) changes from an axisymmetric shape to a non-axisymmetric diamond pattern as R/t changes from small to large values (reduced wall thickness), as shown in Fig. 6.21. In addition, as the wall thickness is reduced, the buckling load decreases and the load–displacement relationship changes from a yielding type to a bifurcation type as described in Sect. 3.4.4. This is because, in a range of small R/t , the buckling stress exceeds the yield stress, resulting in apparition of plastic buckling. As for the buckling of a cylinder subjected to axial compression, the buckling load obtained from experiments becomes extremely lower than the theoretical value in some cases, and this is known to be mainly due to initial imperfections.

For the buckling of a cylinder subjected to uniform bending, there are two types of buckling: buckling that leads to the reduction in load capacity due to the ovalization (flattening) of the cross section, and the local buckling of a shell. In a region of small R/t , limit point (flattening instability), buckling due to the ovalization of cross section is mainly found, while, in a region of large R/t , local (bifurcation) buckling becomes dominant. Between those two regions, a combination of both types of buckling can occur. In the case of the buckling of a cylinder subjected to bending, since the initial imperfection that reduces the buckling stress observed in experiments has the effect only at the point where maximum stress occurs, the possibility of the reduction is less than when the axial compression load acts on all regions.

Fig. 6.21 Buckling modes of a cylinder subjected to axial compression or bending load



[3] Design concept for buckling

Buckling behavior relates to time-dependency as well as to loading characteristics, as described in Sect. 3.4.4 in Chap 3. Therefore, considering these two factors, buckling modes are classified according to Table 6.6. Therefore, since the post-buckling behavior differs significantly depending on the buckling mode, appropriate safety margin must be provided according to each mode. Indeed, concerning the time-independent buckling, a sufficiently large design factor is required because load-controlled buckling does not reduce the load even after buckling and is likely to lead to sudden failure. On the other hand, for the strain-controlled buckling, a smaller design factor is considered acceptable as strain-controlled buckling maintains an equilibrium state even after buckling. It is therefore more unlikely to lead to sudden failure.

Concerning the time-dependent buckling, since the nonlinearity between stress and creep strain is large, and in the case of load-controlled buckling, it is possible to provide a safety margin equal to or greater than 10 with regard to critical time by taking a design factor of approximately 1.5 with regard to load. For the strain-controlled buckling, since the load is moderated along the increase in deformation,

Table 6.6 Classification of buckling modes and applicable design factors

Buckling modes		Design factor (service condition A)
Time-independent buckling	Load-controlled buckling	3 (for load)
	Strain-controlled buckling	1.67 (for strain)
Time-dependent buckling	Load-controlled buckling	1.5 (for load)
	Strain-controlled buckling	1.0 (for strain)

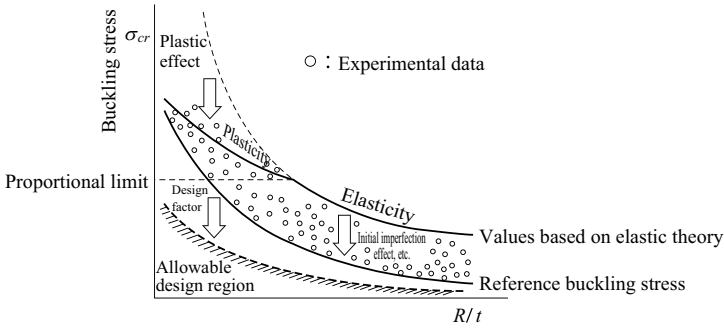


Fig. 6.22 Design concept for buckling of a cylinder subjected to axial compression or bending load

sudden failure is unlikely to occur. Based on these considerations, the design factor applied to buckling is provided in the design rule for each buckling mode, as described in Table 6.6 [2].

In addition to the classification shown in Table 6.6, the buckling mode can be further reclassified from such perspectives as elastic and plastic buckling, axial compression and bending. Furthermore, the buckling load decreases due to initial imperfections generated from manufacturing inaccuracies, and the degree of the decrease depends strongly on the buckling mode. Accordingly, studies are in progress to assess and prevent buckling with an appropriate safety margin, considering the effect of initial imperfections for each buckling mode. Figure 6.22 is an example of the basic design concepts for buckling based on the analysis of a test result on the buckling of a cylinder subjected to axial compression or bending load (Fig. 6.21). First, for a perfect cylindrical shell, the theoretical value of the elastic buckling, or the theoretical solution which considers plasticity when the theoretical value exceeds the proportional limit, is used as the basis. Experimental data are normally distributed in a range significantly lower than the above-noted theoretical solution. Since the difference with the theoretical solution is due to the initial shape imperfections, beside the variation in material properties, the reference buckling stress σ_{cr} is determined on the assumption of appropriate initial imperfection to envelop the lower limit of the experimental data. The allowable design stress is specified by introducing the design factor f_B to the reference buckling stress.

6.4.6 Strength Influencing Factors and Consideration in Design Evaluation

The limiting values for each failure mode described above have been, in general, developed based on the data obtained from uniaxial testing of base material in the

air. In actual plants, on the other hand, the influencing factors exist, and may significantly affect design depending on the service conditions and materials. Therefore, the applicable scope of a design evaluation method is focused on a range where the influencing factors do not significantly affect the design. Additional design factors are sometimes introduced when appropriate.

[1] Welded joint

As observed in detail, a welded joint is a discontinuous structure consisting of base metal, weld metal, and a heat affected zone (HAZ), as well as a weld bead depending on the welding method. Therefore, the strength of the welded joint may be reduced due to the strength of each portion, such as weld metal, and the interaction among portions with different characteristics.

For example, the use of a softer weld metal with a priority on ductility leads to a structure in which a relatively softer weld metal is sandwiched between base metal zones. When cyclic loading is applied to this weld zone, strain concentration (metal-lurgical notch) may appear as the sections possess different mechanical properties from one another. When taking the HAZ into account, the situation above becomes even more complicated. Relative strength and hardness among the base metal, weld metal, and the HAZ differ depending on the material and welding method.

Therefore, various methods based on test data and strain concentration mechanisms have been proposed to evaluate the creep-fatigue strength of a welded joint. As an example of the method adopted in an official code, an evaluation method for welded joints using the fatigue strength reduction factor and the creep strength reduction factor for specified materials is provided in Europe [12]. In Japanese design rule, the limiting strain value for base metal is reduced by half when applied to welded joints.

[2] Sodium environmental effect

It is known that, in high-temperature air, the fatigue strength is decreased by the effect of oxidation and the amount of strength reduction due to oxidation becomes less significant when creep is dominant. This is because the formation of oxidized scales accelerates the occurrence of surface cracks due to fatigue, even though it does not directly affect the cavity creation at the grain boundary that controls creep. On the other hand, in sodium, whose oxygen concentration is controlled to a low level, the effect of sodium corrosion is insignificant and a similar tendency to that in vacuum is found unless the temperature exceeds 600 °C.

In addition, decarburization or carburization of steel may occur in sodium. Decarburization reduces the creep strength and at the same time reduces the stress generated because of softening. Carburization reduces ductility and suppresses plastic deformation because of hardening. Since these phenomena have conflicting effects on strength and occur in a region extremely close to the wall surface, it was verified by experiment that they do not become a factor that significantly reduces the lifetime under plant operation conditions. From the above, the evaluation method for creep-fatigue based on the data obtained in the air provides conservative results for ordinary components exposed to sodium.

[3] Neutron irradiation effect

When neutron energy exceeds 0.1 MeV, atoms in a metal crystal lattice get knocked out by collision with neutrons, leading to disturbance of the crystal lattice (i.e., lattice defects). These defects hinder the movement of dislocations, leading to a reduction in ductility due to hardening. Therefore, the design rule specifies the upper fluence limit of neutrons ($E > 0.1$ MeV) according to the temperature and creep effect (e.g., 5×10^{25} n/m² for base metal at 400 °C) as a condition to allow application of the strain limits and material strength criteria to 304SS without any specific correction. Because the mechanical characteristic most sensitive to the neutron irradiation effect is the rupture elongation and because the design rule assumes that the materials used are to be ductile, these limiting values are specified to ensure sufficient ductility of the material.

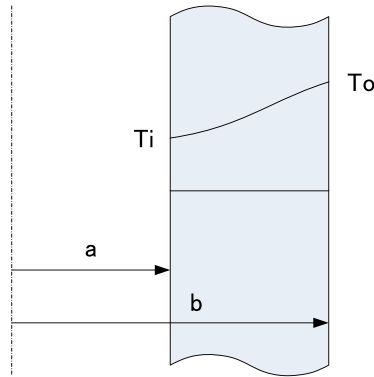
Besides, thermal neutrons convert minute impurities in steels into helium (He). At temperatures where creep occurs, reductions in ductility and creep strength are caused, because the generated He atoms enhance the aggregation of vacancies into the grain boundary, etc. Accordingly, when the fluence of thermal neutrons ($E < 0.4$ eV) exceeds a specified value (10^{19} n/m²), the correction factor that depends on the fluence is introduced to the limiting values related to the creep rupture time and creep strain rate.

[4] Thermal aging

When a metallic material is exposed to high temperatures, structural changes including formation and coarsening of deposits (carbide, etc.) and transformed phases (Laves phase, etc.) may occur. As a result, changes in the mechanical properties, such as increases in static tensile strength and yield stress, softening of cyclic stress–strain properties, and reductions in ductility and toughness, are likely to occur. It is necessary to appropriately take these changes into consideration as extended plant lifetimes can cause these changes in mechanical properties to significantly affect the assumed failure modes.

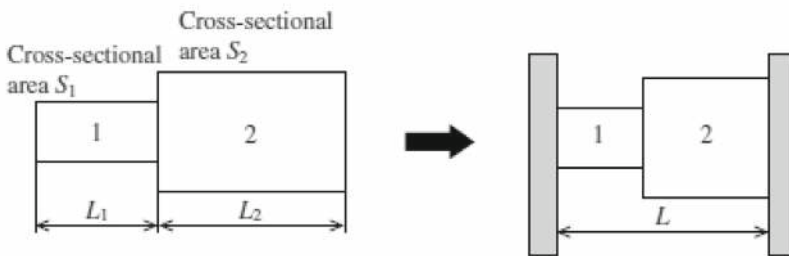
Exercises

1. Describe the differences between LWRs and FRs regarding the structural design condition and the assumed failure modes.
2. Concerning a cylinder whose inner and outer surface temperatures are kept at T_i and T_o , respectively, as shown in the figure below, answer the following questions.
 - (1) With inner diameter a and outer diameter b , derive equations that provide temperature and stress distributions in the radial direction.
 - (2) Based on the above equations, derive an approximate equation for the case of a thin-walled cylinder having wall thickness t .



Cylinder whose inner and outer surfaces are subjected to different temperatures

3. Suppose that a stepped bar consisting of two members, members 1 and 2, having Young's modulus E [Pa] is fixed at its both ends to two rigid walls that are separated by distance L [m] ($L < L_1 + L_2$), where L_1 (m) and L_2 (m) are the lengths and S_1 [m²] and S_2 [m²] are the cross-sectional areas of members 1 and 2, respectively. Answer the following questions.
 - (1) Obtain the stress occurring at each member of the bar.
 - (2) Obtain the stress occurring at each member of the bar when the temperature rises by ΔT [K] while the stepped bar is fixed to the rigid walls. Here, let the linear expansion coefficient of the bar be α [K⁻¹].
 - (3) How does the bar behave if creep occurs under the condition of (2)? Describe the behavior and explain the reason why you think so.
 - (4) Answer the name of the phenomenon that occurs in (3).



Stepped bar fixed at its both ends

4. Concerning the following structures, describe the characteristics of loads and the failure modes to which the greatest attention must be paid together with the reasons.

- (1) Low-pressure vessel made of 304SS subjected to cyclic temperature change between room temperature and 500 °C;
- (2) High-pressure vessel made of 304SS subjected to cyclic temperature change between room temperature and 500 °C;
- (3) Thin-walled vessel made of 304SS subjected to an external pressure at 500 °C.

Answers to Exercises

1. (omitted)
2. (1) The temperature distribution in a case where there is a steady-state thermal flow with which the inner surface temperature and the outer surface temperature are maintained at constant values T_i and T_o , respectively, is given by

$$T = T_i - \frac{\ln(r/a)}{\ln(b/a)}(T_i - T_o)$$

The stress in this state is given by

$$\begin{aligned}\sigma_r &= \frac{\alpha E(T_i - T_o)}{2(1-\nu)\ln(b/a)} \left\{ -\ln \frac{b}{r} - \frac{a^2}{b^2 - a^2} \left(1 - \frac{b^2}{r^2} \right) \ln \frac{b}{a} \right\} \\ \sigma_z &= \frac{\alpha E(T_i - T_o)}{2(1-\nu)\ln(b/a)} \left\{ 1 - 2\ln \frac{b}{r} - \frac{2a^2}{b^2 - a^2} \ln \frac{b}{a} \right\} \\ \sigma_\theta &= \frac{\alpha E(T_i - T_o)}{2(1-\nu)\ln(b/a)} \left\{ 1 - \ln \frac{b}{r} - \frac{a^2}{b^2 - a^2} \left(1 + \frac{b^2}{r^2} \right) \ln \frac{b}{a} \right\}\end{aligned}$$

If $T_i < T_o$, σ_θ represents tensile stress on the inner surface and compressive stress on the outer surface. σ_r represents 0 on the inner and outer surfaces and tensile stress inside the wall.

If $T_i > T_o$, the reverse applies. σ_θ on the inner surface represents the maximum stress.

- (2) If the temperature distribution inside the wall is assumed to be linear, the following equation can be applied.

$$\sigma_\theta = \sigma_z \pm \frac{\alpha E}{2(1-\nu)}(T_i - T_o)$$

Here, “+” provides a value for the outer surface and “−” a value for the inner surface.

3.

$$(1) \text{ Stress of bar 1: } \sigma_1 = \frac{ES_2\{L - (L_1 + L_2)\}}{L_1S_2 + L_2S_1} \quad [\text{Pa}]$$

$$\text{Stress of bar 2: } \sigma_2 = \frac{ES_1\{L - (L_1 + L_2)\}}{L_1S_2 + L_2S_1} \quad [\text{Pa}]$$

$$(2) \text{ Stress of bar 1 } \sigma_1 = \frac{ES_2\{L - (1 + \alpha\Delta T)(L_1 + L_2)\}}{L_1S_2 + L_2S_1} \quad [\text{Pa}]$$

$$\text{Stress of bar 2: } \sigma_2 = \frac{ES_1\{L - (1 + \alpha\Delta T)(L_1 + L_2)\}}{L_1S_2 + L_2S_1} \quad [\text{Pa}]$$

(3) In bar 1, compressive strain increases as stress is relaxed. In bar 2, compressive strain decreases as stress is relaxed.

(4) Elastic follow-up.

4. (omitted).

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Chapter 7

Seismic Design



In Japan, a world prominent earthquake-ridden country, earthquakes have been inevitably regarded as the external events (so-called external initiating events) impacting nuclear power stations. In the meantime, the structural design described in Chaps. 5 and 6 considered internal events (i.e., internally caused initiating events) taking place inside nuclear power stations. Efforts can be made to reduce the occurrence of events resulting from internal factors. However, with regard to the externally caused initiating events, there is no other way to prevent the occurrence than selecting sites. In addition, it is difficult to predict the occurrence of earthquakes, and upon occurrence, the entire components within a nuclear power station would be affected. Therefore, it is necessary to consider safety issues on the assumption that earthquakes occur without choosing the time and place. In order to assure structural integrity, viewpoints are required for seismic design that somewhat differ from those for internally caused initiating events.

In the first place, when considering risks pertaining to nuclear power stations, what earthquake intensity (more precisely, earthquake ground motion) should be taken into account to ensure safety? As noted also in Chap. 5, while it is essential that buildings, components, etc. should be designed rationally in according with their importance, if we consider that an earthquake is a common initiating event, what is the significance that we need to focus on and how should we classify it? In addition, supposing that earthquakes do not choose the time and place, what are the situations of a nuclear power station for which we need to be prepared against the occurrence of an earthquake? Namely, during what internal initiating events do we need to postulate the occurrence of an earthquake? Furthermore, what are the situations we need to postulate as internal initiating events that would be induced when an earthquake occurs? In order to ensure safety, what are the conditions of the components that need to withstand the loads applied at that time? This chapter addresses these issues. Regarding what we must take into account in seismic design, the regulatory guide for reviewing seismic design and other related guides serve as a useful reference, in which the Nuclear Safety Commission of Japan and the Nuclear

Regulation Authority that took over it in 2012 summarized the points for reviewing design adequacy from the viewpoint of seismic safety assurance. Therefore, this chapter focuses on this Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities, referring to other related guides.

History shows that the seismic design of nuclear power stations has progressed and become more sophisticated along with the development of nuclear industry. As briefly mentioned in Chap. 1, in March 2011, TEPCO (Tokyo Electric Power Company, Inc.) Fukushima Daiichi (Daiichi: literally, the first, or No.1) Nuclear Power Station, which is located along the coast in Fukushima Prefecture, was attacked by the 2011 off the Pacific coast of Tohoku Earthquake (hereinafter, referred to as the “2011 Tohoku-Oki Earthquake”) and the tsunami triggered by this earthquake, and consequently had a severe accident. The moment magnitude of this earthquake was M_w 9.0. We have learned many lessons from this severe accident, and utilized for drastically changing the safety regulations in Japan, in particular, the safety regulatory framework as well as regulatory guides relating to safety reviews on tsunami-resistant design.

First, the basic concept which constitutes the basis for the seismic design of nuclear power stations is described in Sect. 7.1. Section 7.2 explains how the concept of seismic design has historically progressed, and Sect. 7.3 describes the concept of the Regulatory Guide for Reviewing Seismic Design which was revised in 2006 on the basis of the latest knowledge at that time and as a culmination of what had been done for strengthening the seismic safety regulation. Section 7.4 explains how the seismic and tsunami-resistant designs were strengthened as drastic changes in safety regulations in response to lessons learned from the severe accident that occurred at the Fukushima Daiichi nuclear power station (hereinafter, referred to as the “Fukushima Daiichi nuclear accident”). Lastly, Sect. 7.5 briefly discusses actual cases of seismic design.

7.1 Basic Concept

Basically, in seismic design, we first comprehend the degree of safety each nuclear power station must achieve (i.e., how safe is safe enough?), estimate the level of potential earthquake ground motions that need to be postulated for the degree of safety, consider the required conditions of structures, components, etc. when they withstand the seismic forces exerted by the earthquake ground motions, and design the facilities, components, etc. on the basis of these considerations.

With regard to the seismic design of nuclear power stations, in order to meet one of the international safety requirements (IAEA Safety Series No.50-C-S (Rev.1)) stating that “the radiological risks associated with the external events should not exceed the range of the radiological risks associated with the accidents of internal origin,” it is fundamental that the design be conducted postulating earthquakes exceeding the severest earthquake that can be reasonably expected to be experienced at the site area during the operation life of a nuclear power plant. Accordingly, not to mention the historically severest earthquake to be expected, even if a large-scale earthquake of a

very low occurrence probability postulated from scientific knowledge and exceeding the severest earthquake to be expected to be experienced during the operational life occurs, the earthquake shall not trigger the dispersion of a large amount of radioactive materials.

However, upon the occurrence of a large-scale earthquake, it is not realistic to design in such a way that all structures of the nuclear reactor facility would not be damaged at all. The design should be optimized from the viewpoints of both safety and economy, with safety being the most important. Therefore, it is rational to define the significance (importance in the seismic design) based on the degree of damage inflicted on each piece of buildings, structures, equipment and components constituting each facility due to earthquakes, the quantity of radioactive materials released thereby, and the radiological consequences arising from these radioactive materials on the general public, and then classify the equipment, etc. and perform seismic design in accordance with their importance. Here, the seismic design in accordance with the importance means, as noted above, that the design is conducted in such a way that the risks due to an earthquake ($=$ probability of the earthquake occurrence $\times$ magnitude of radiological consequences caused by the earthquake) may not exceed the risks due to the internal origins. Specifically, this means that facility, buildings, structures, equipment and components that would not cause radiological consequences in the case of their breakage shall be handled by the seismic design equivalent to that for facilities in general industry. In contrast, if facility, etc. would exert extensive radiological hazard on the general public in the case of their breakage, more intense earthquake shaking the relevant site than postulated for facilities in general industry shall be assumed, even if the probability of its occurrence may be extremely small, and the equipment and components shall be designed in such a way that they can withstand and prevent accidents induced by the earthquake. This is conceptually illustrated in Fig. 7.1. This seismic design offers a means to prevent accidents that may potentially occur in nuclear power stations in the case of an earthquake and to carry out measures for mitigating the consequences appropriately even if the earthquake occurs during an accident, thereby keeping the risks to the general public sufficiently small.

In terms of the magnitude of radiological consequences, i.e., the extent of hazard, it could be classified into the following levels:

- (a) Even if the relevant facility, buildings, structures, equipment, and components are failed, their continuous operation will be possible, or even if they need to be repaired, they will not cause radiation hazards to the workers and the general public.
- (b) If the relevant facility, buildings, structures, equipment, and components are failed, radioactive materials will be released excessively within the facility (power station), causing exposure of mainly workers to radiation. This accompanies stop of operation for repair and decontamination.
- (c) If the relevant facility, buildings, structures, equipment, and components are failed, radioactive materials will be released excessively to the area surrounding the power station, potentially causing radiation hazards to the general public. (If

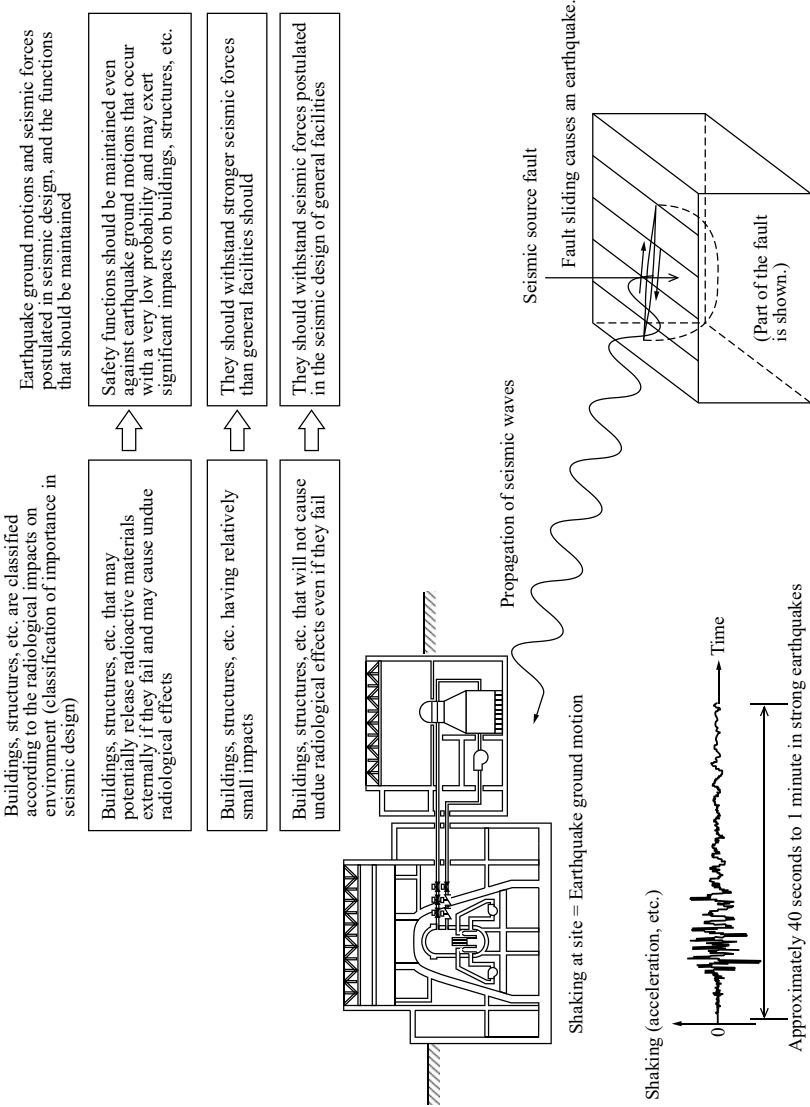


Fig. 7.1 Classification of importance in seismic design and concept of postulated earthquake ground motion

the effective dose to the general public in the surrounding area does not exceed 5 mSv, the influence of the exposure can be regarded as small.)

Conceptually, the seismic design of the facility, buildings, structures, equipment, and components corresponding to the level (a) may be on the same level as that for facilities in general industry, and such facilities, etc. are referred to as Seismic class C facilities in the current Japanese seismic design.

Similarly, those corresponding to the level (b) and (c) are referred to as Seismic class B and class S, respectively. Such a classification according to the radiological consequence (i.e., focusing on the magnitude of radiological consequences) is referred to as the classification of importance in the seismic design. In the meantime, although the basic concept of the classification of importance in the seismic design has not been changed substantially, Seismic class S has historically experienced transition in the details of classification and definition. The above-mentioned concept of the classification of importance in seismic design is applied to the “facilities for design basis events” that cope with design basis events including design basis accidents as described in Chaps. 1 and 5. The corresponding concept to be applied to the “facilities for coping with severe accidents, etc.” is explained in Sect. 7.4. Note here that severe accidents, etc. include not only severe accidents but also beyond design basis accidents with the possibility of escalating to severe accidents, as defined in Chap. 1. Next, with regard to the seismic forces (seismic loads) that the facilities in each class shall withstand, it is desirable that, on the basis of the concept of the risks mentioned above, the probability of earthquake occurrence can be anticipated to some extent and the maximum seismic loads acting on facilities, etc. can be predicted as well. However, the magnitude has been decided deterministically in the past seismic design as follows. Namely, in Seismic class B, it is stipulated that design shall be conducted by applying one level higher earthquake resistance compared to Seismic class C, and the seismic forces (static seismic forces, in this case) shall be 1.5 times higher than those for Seismic class C. For facilities corresponding to Seismic class S of the present classification, it was stipulated that utmost attention shall be paid in the design because they are relevant to radiation hazards to the general public, earthquake ground motions with very low probabilities of occurrence but to be expected to occur during the operational life shall also be assumed, and such ground motions shall be obtained from dynamic analysis. However, since seismic forces become small in low seismicity areas in the dynamic analysis, the lowest limits of seismic forces shall be set in order to prevent this reduction. These values shall be given as the static seismic forces, which for the horizontal seismic intensity shall be three times as large as the standard seismic intensity prescribed by the Building Standards Act on the basis of experiences. With the revision of the Building Standards Act in 1981, the static seismic force is determined in accordance with the seismic story shear force coefficient method (or simply, story shear coefficient method. Or, repeatedly mentioned, earthquake layer shearing force coefficient) in place of the so-called seismic intensity method. Since the occurrence frequency of earthquakes has been on the rise in recent years and unexpected earthquake ground motions have occurred, sophisticated evaluation methods are being developed.

Mentioned above are the basic concept of the conventional seismic design in Japan, which was applied before the Fukushima Daiichi nuclear accident. This basic concept has been still applied as the concept for the “facilities for design basis events,” that is, has been applied to perform functions required as safety measures for the first to third stages of the defense-in-depth safety design approach. The corresponding concept for the “facilities for coping with severe accidents, etc.” is described in detail in Sect. 7.4. In short, before severe accidents, etc. occur, important functions necessary to perform at severe accidents, etc. should not be lost even if an earthquake to be expected to be experienced during the operational life, namely, identical to a design basis earthquake ground motion S_s to be applied to Seismic class S facilities happens. After severe accidents, etc. occur, the earthquake ground motion for seismic assessment after the accident should be appropriately established, taking into consideration its occurrence probability.

The concept of seismic design differs depending on countries. In the U.S., items (b) and (c) in the above-mentioned classification of importance in the seismic design are referred to as category 1, and item (a) is defined as other than category 1. Furthermore, in category 1, dynamic analysis is basically used and it is required that safety functions be ensured against the seismic force caused by a safe shutdown earthquake (SSE).

Before the Fukushima Daiichi nuclear accident, on the basis of the above-mentioned basic concept, for ensuring safety for nuclear power stations (nuclear power reactor facilities) specific prescriptions on the “classification from the viewpoint of seismic design,” “appropriate design seismic forces,” and “design to bear the design seismic force sufficiently” were stipulated in the “Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities” (approved by the Nuclear Safety Commission of Japan, September 2006, abbreviated to “Seismic Design Review Guide”). These prescriptions were given in the “classification of importance in seismic design,” “determination of the design basis earthquake ground motion,” “principle of seismic design” (the design policy in accordance with each seismic class), “load combination and allowable limit,” and the newly added “consideration of the accompanying events of earthquake.” After the Fukushima Daiichi nuclear accident, the Nuclear Regulation Authority (NRA) stipulated prescriptions in the Ordinance on Standards for Location, Structures and Equipment and the related regulatory guides, similar to those mentioned above. In the Ordinance, prescriptions on countermeasures against tsunamis are given independently on those against earthquakes, because tsunamis are not always generated seismically.

In the stage of the installation licensing (so-called safety review) of a nuclear power station, the government reviews, referring to regulatory guides such as the above-mentioned Seismic Design Review Guide, about whether the basic design and design guidelines—the most important aspect in ensuring the safety of the facility—of the application submitted by the licensee are adequate. Subsequently, in the process of approving the design and construction plan, the validity of detailed specifications such as performance, structural strength, etc. that each piece of facilities, etc. should possess will be confirmed, paying attention to the consistency with the basic design and design guidelines approved in the installation licensing stage.

7.2 Transition in the Concept of Seismic Design

In seismic design, as described in Sect. 7.1 (see Fig. 7.1), buildings, equipment, etc. are classified in accordance with the degree of anticipated radiological hazards (classification of importance in the seismic design), and the levels of large-scale earthquakes that need to be postulated are determined considering the degrees of radiological hazards that nuclear reactor facilities may cause and assuring that the general public would not be exposed to undue radiation risks. Then, the seismic forces exerted on buildings, equipment, etc. due to the ground motions induced at the relevant site by these earthquakes are derived, situations of the facility to be maintained during and after the earthquake are established, and allowable limits are determined to guarantee to maintain these situations. Then, the facilities are designed to meet the allowable limits.

On the basis of this basic concept, the present section summarizes briefly the historical background as to how the seismic importance classification, postulated earthquake ground motions, allowable limits, etc. have been conceived. This is because the current Regulatory Guide for Reviewing Seismic Design has not been developed abruptly, but it has been constructed on the basis of the past experience with continuous efforts toward sophistication reflecting the latest knowledge on earthquakes. The historical background of the seismic design for nuclear power stations has been systematically summarized in Ref. [1]. In reference to this literature, this section describes at first how the seismic design had been conceived in the earliest days when nuclear power stations were introduced and domestic production had started in Japan. Then, we look back on the time when the study on the standardization of seismic design had made substantial progress in the decade starting from 1965 that witnessed a construction boom. These efforts were integrated and reflected in the “Seismic Design Review Guide” or “Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities” instituted in 1978 and revised in 1981. In 2006, this regulatory guide was completely revised, because since 1981. Seismology and earthquake engineering have been greatly developed, and large-scale earthquakes such as the Southern Hyogo Prefecture Earthquake that occurred in January 1995 and the Western Tottori Prefecture Earthquake in October 2000 were experienced. Damages due to these strong earthquakes to nuclear reactor facilities (for example, the Oi Nuclear Power Station and Shimane Nuclear Power Station) were insignificant. Afterward, however, the Niigata Chuetsu-Oki (off the Chuetsu coast) Earthquake that occurred in July 2007 and the 2011 Tohoku-Oki Earthquake mentioned at the beginning of this Section caused damages to several nuclear power plants. In particular, core meltdowns occurred in the Unit 1 to 3 of the Fukushima Daiichi Nuclear Power Station, which were triggered by the tsunami caused by the 2011 Tohoku-Oki Earthquake. These core meltdowns resulted in release of a large amount of radioactive materials to the environment. This section gives discussions of up to the 1981 Seismic Design Review Guide. In Sect. 7.3, the 2006 Seismic Design Review Guide is introduced. Section 7.4 addresses the regulatory guide on seismic and tsunami-resistant design that was significantly strengthened in response to lessons learned from the Fukushima Daiichi nuclear accident.

7.2.1 *The Earliest Days*

The Japan Atomic Power Company (JAPC) constructed Tokai Power Station as the first nuclear power station in Japan at Tokai Village, Ibaraki Prefecture. Its installation license was applied for in March, 1959. A British Calder Hall type reactor was introduced for this nuclear reactor. Hence, the reactor core was configured with piled-up graphite blocks (referred to as bricks). Since Japan is an earthquake-ridden country unlike the UK, aseismic countermeasures have been the most critical issue. Key points in seismic design summarized specifically for this station, as listed below, have constituted the conceptual basis for the seismic design of the ensuing nuclear power stations. The following description refers to Refs. [2, 3].

- (a) For various facilities of a nuclear power station, the higher the importance or risks to the general public are, the more stringent seismic design should be conducted. From the viewpoint of the yardstick of social disasters, there is an idea of classifying the relative importance in accordance with the radiological consequences on the general public in the case of failure due to an earthquake. On the other hand, from the viewpoint of owner's damages, there is an idea of making classification in consideration of reparability and structural characteristics—such as the damage is extremely difficult to repair although it will not lead to a disaster. In Tokai Power Station, the importance classification was conducted in consideration of both ideas, specifically as follows.

Most important facilities: The central part of the nuclear reactor section (nuclear reactor, primary cooling system, safety equipment systems, etc.)

Important facilities: the peripheral part of the nuclear reactor section (secondary cooling system, radioactive materials handling equipment system, etc.)

General facilities: The power generation section (general power generation equipment systems etc.)

- (b) The rigid structure method should be adopted because of the following reasons: rigid structures are much easier to design than flexible structures, competition with thermal deformation can be avoided by adopting adequate supporting structures, the design method is simple and easy to be understood by British designers, etc.
- (c) As an approach for seismic design, it was decided that the static seismic intensity method prescribed in the Building Standards Act of those days should be used. This was because the experience of the seismic design based on the dynamic analysis (the dynamic seismic design) had been scarce in those days, and also it was considered that the static seismic intensity method could sufficiently ensure safety if the structure as a whole was rigid. However, in consideration of the significance of seismic design for a nuclear power station, it was decided that dynamic analysis should be added as an assessment method for owner's reference.
- (d) Regarding the static seismic design, it was considered that, so long as the design for the most important structural portions of the reactor was conducted assuming

the value around 0.6 g as the maximum earthquake ground motion anticipated in Tokai Village, a sufficient safety margin could be secured even if the amplification due to the ground and the response of the structures were taken into account. Therefore, it was decided that three times the seismic coefficient value specified in the Building Standards Act, corresponding to 0.6 g , should be assumed for these most important structural portions. Here, as will be discussed in Sect. 7.5, the seismic intensity represents the ratio of the seismic acceleration to gravitational acceleration g . The design seismic intensity that should be assumed for each portion was therefore set as indicated in Table 7.1. Here, C_0 is the standard design seismic intensity based on the Building Standards Act. Although C_v is supposed to have the same numerical value as C_0 , it is a design seismic intensity in the vertical direction with a condition that it acts concurrently with a horizontal seismic intensity in an adverse direction.

- (e) It was decided that a seismic switch be installed in a manner such that an automatic scram occurs if the horizontal response acceleration in the periphery of the core exceeds the prescribed value.
- (f) With regard to the buildings, the allowable stress during an earthquake should be based on the Building Standard Law Enforcement Order. With regard to the components and piping systems, the values should be less than the yield stress in their service conditions (Ref. [3], p. 120).

Following Tokai Power Station, the nuclear power stations of Tsuruga Unit 1 (BWR) of JAPC, Mihama Unit 1 (PWR) of Kansai Electric Power Co., Inc. (KEPCO), and Fukushima Daiichi Unit 1 (BWR) of Tokyo Electric Power Co., Inc. (TEPCO) introduced the dynamic seismic design positively from the recognition that the seismic design based on the dynamic approach was indispensable to important facilities like nuclear power stations where the assurance of high level of safety was required. In addition to this, the following advancement beyond the concept of Tokai Power Station was adopted in the seismic design of these power stations.

Table 7.1 Design seismic intensities set to the JAPC Tokai power station [1, 2]

Seismic importance classification		Design seismic intensities in the static seismic intensity method	
Classification	Corresponding facilities	Buildings	Mechanical systems
Most significant facilities	Reactor primary cooling system, reactor safety protection systems, emergency power system, etc.	$3C_0 \pm 1.5C_v$	
Significant facilities	Secondary cooling system (heat exchanger-related), waste facilities, etc.	$1.5C_0$	$1.5C_0 \pm 0.75C_v$
General facilities	General electric power facilities, etc.	$1.0C_0$	

[Note]

C_0 : Standard design seismic intensity based on the Building Standards Act of Japan

C_v : This shall be the seismic intensity in the vertical direction with its value identical to C_0 .

- (a) The importance was classified into As, A, B, and C in Tsuruga Unit 1. This concept was inherited to the Seismic Design Review Guide instituted in 1978 and modified in 1981.
- (b) In setting the maximum acceleration amplitude of a design earthquake ground motion, the maximum earthquake ground motion that may actually occur at the site during the plant life should be considered. Then, the facilities in Seismic classes As and A should be designed in such a way that, when this design earthquake ground motion is applied, the stress may fall within the yield stress as a whole (gross elastic design).
- (c) Furthermore, for facilities belonging to Seismic class As that are particularly critical for ensuring safety, it was decided that the maximum earthquake ground motion conceivable at this site should be considered and that the required functions should be ensured during its occurrence. This earthquake ground motion should be 1.5 times the intensity of the design earthquake ground motion. It can be said that these design earthquake ground motion and hypothetical earthquake ground motion (or the earthquake ground motions for safety margin check) heralded the concept of the design basis earthquake ground motions S_1 and S_2 described in the “Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities, 1981 edition.” However, the notion of those days was that the design should be conducted absolutely with the design earthquake ground motion, and the hypothetical earthquake ground motion was directed for safety assessment regarding the assurance of safety functions. This marks a difference from S_1 and S_2 , which were both design basis earthquake ground motions.
- (d) In Mihama Unit 1, simulated earthquake ground motions considering the characteristics of the site were generated, and the design earthquake ground motion was defined as a spectrum. The definition of the spectrum will be given in Sect. 7.7.3.
- (e) In the meantime, the finite element analysis (FEA) was used for the first time as a seismic design technique.

7.2.2 Systematization as Regulatory Guides

From 1969 through 1977, as many as 24 units of power stations were designed and constructed. Innovative techniques were successively adopted in seismic design. Meanwhile, many vibration tests were conducted to verify the validity of previous seismic design methods and a number of new findings were accumulated, contributing greatly to the progress of the seismic design technology.

Responding to the design and construction activities pertaining to such nuclear power stations, the government also gained momentum in the activity toward formulation of the seismic design standard. This effort had resulted in the institution of the 1978 Seismic Design Review Guide by the Nuclear Safety Commission of Japan. As repeatedly mentioned, this 1978 edition was revised as the 1981 edition, reflecting

the revisions of the Building Standards Act. The detail of the revision of the Guide is described below.

The activities from the earliest days can be summarized in chronological order as follows.

- 1957: The Earthquake Countermeasures Subcommittee was organized in the Atomic Energy Commission of the Prime Minister's Office. While performing investigation studies in preparation for the introduction of a Calder Hall type nuclear reactor, the Subcommittee carried out the investigation to establish the basic policy for seismic design.
- 1961: the Nuclear Power Station Safety Standards Committee of the Ministry of International Trade and Industry published the "1st Report on the Nuclear Power Station Safety Standard." The seismic design policy was summarized in Article 19 of this document.
- 1964: The "Regulatory Guide for Reviewing Nuclear Reactor Siting Evaluation and Application Criteria" was instituted.
- 1965: The "Technical Standards on the Nuclear Power Generation Equipment" was instituted.
- 1966: The Earthquake Countermeasures Subcommittee of the Nuclear Power Station Safety Standards Committee studied specifically on the standardization of the dynamic approach, and summarized the "Investigation Report on Seismic Design of Nuclear Power Stations." In this report, the importance classification was conducted for each facility and specific examples were indicated. In addition, the concept was clearly stated that accidents and earthquakes should be handled independently because seismic design aims to prevent the possible occurrence of accidents due to earthquakes and differs from the safety assessment in which the occurrence of accidents is presupposed. With regard to the dynamic analysis, the difficulty of establishing the standard for the dynamic design method was pointed out considering the circumstances of those days.
- 1978: The "Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities" was compiled as a result of efforts by the Review Committee on Seismic Design, etc. in the Nuclear Safety Commission of Japan. As will be detailed below, the importance classification in seismic design was clarified in this document, and the method for assessing seismic design, load combinations and the allowable limits of stress were specified.

Picking up only the key points of the method for evaluating seismic design, it was decided that the static and dynamic analyses should be used in combination for the facilities in Seismic classes A and As. A new concept was introduced in earthquake ground motions for use in the dynamic analysis. Namely, the free surface of the base stratum was defined as the hypothetically exposed surface of the base stratum that would emerge as a result of removing the surface layer, the undulation in the earth's surface and the structures themselves altogether from the site, and the design basis earthquake ground motions were defined on this surface. It was stipulated that the S_1 earthquake (strongest design earthquake) and the S_2 earthquake (critical design

earthquake) should be selected as two types of earthquakes that cause the design basis earthquake ground motions.

In addition, existing power stations were assessed on the satisfaction of this new review guide (so-called the back-check). Because fairly large margins were expected regarding the intensity of seismic force and the structural safety factor, it could be confirmed that the seismic safety was ensured sufficiently.

- Also, in 1978: The Committee of Experts on Nuclear Reactor Safety of the Nuclear Safety Commission of Japan published the “Guidelines for Safety Review Concerning Geology and Ground of Nuclear Power Stations.”
- 1981: The Nuclear Safety Commission of Japan revised the 1978 edition of Seismic Design Review Guide.

The Building Standards Act was revised so as to update a method for determining static seismic forces to the story shear force coefficient method in place of the seismic intensity method, and others. Reflecting these revisions, the 1978 edition was revised in 1981. In the following, this revised version in 1981 is referred to as the 1981 Seismic Design Review Guide (the legal name is the “Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities, 1981 edition”).

In this review guide, in consideration of the status of the seismic design technology of those days, it was clearly stated in the preface that “this guide was instituted on the basis of the engineering judgment of the knowledge on seismology, geology, etc. ... further revision will be necessary in future, as required from the accumulation of new knowledge and experience.” Then, with the advent of a period of brisk seismic activities, lessons were learned from large-scale earthquakes such as the Southern Hyogo Prefecture Earthquake in 1995, and the Western Tottori-ken Earthquake in 2000, the guide was reviewed for sophistication, and a revised edition was instituted in 2006 (this is referred to as the “Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities, 2006 edition” as the legal name. Hereinafter, referred to as the “2006 Seismic Design Review Guide.”).

- 1982: The internal regulation (or by-law) on the standard of seismic design was compiled by the Ministry of International Trade and Industry.

Meanwhile, also in the private sector, it was decided that, in line with the enactment of the Electric Utilities Industry Act, the Japan Electric Association would share the investigation on the technical standards as a designated investigation agency, and the Seismic Design Subcommittee of the Committee of Experts on Atomic Energy compiled the “Technical Guidelines for Aseismic Design of Nuclear Power Plants” in 1970.

Note here that you can see from the historical background, the above-mentioned is related to the stages 1–3 safety measures of the defense-in-depth approach, and therefore, addressing the background of the “facilities for design basis events.” As repeatedly mentioned, in response to the Fukushima Daiichi nuclear accident, the regulatory guide was strengthened and significantly changed to include prescriptions for the “facilities for coping with severe accidents, etc.”

7.2.3 ***“Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities, the 1981 Edition”*** **[4, 5]**

The seismic design of nuclear power stations, as inferred from the historical background described so far, aims to prevent the general public from being influenced by undue radiological hazards, even if they encounter large-scale earthquakes. Therefore, depending on the hazard potential inherent in components, or the degree of potential consequences that radiological hazards would inflict on the general public, it is reasonable to change the severity of the design. To this end, the failure probability should be set rationally for each facility as a whole, and it is desirable that the probability of earthquake occurrences that need to be assumed for the design should be set rationally. From such a viewpoint, in the following, a description will be given of the idea on the classification of importance in the seismic design of facilities, the development of the design earthquake ground motions, the combinations of seismic forces with other loads in accordance with the plant conditions during an earthquake, and their allowable limits.

In the meantime, after the institution of the 1981 Seismic Design Review Guide, we have further experienced large-scale earthquakes and accumulated new knowledge on earthquakes. Consequently, the Guide was reviewed and revised as the 2006 Seismic Design Review Guide, wherein some portions of the concept have been taken over from the 1981 Seismic Design Review Guide. The important notions of the revised version are detailed below.

[1] Classification of Importance in Seismic Design

(1) The Basic Concept

In the 1981 Seismic Design Review Guide, it was stipulated that facilities should be classified into Seismic classes A, B and C as follows, from the viewpoint of impacts of radiation to the environment potentially caused by earthquake.

Seismic class A: Facilities that contain radioactive materials in themselves or are related directly to the facilities containing them, potentially diffusing radioactive materials to the environment if their functions are lost, and also facilities required for preventing the occurrence of those events, as well as facilities required for reducing the consequences resulted from the diffusion of radioactive materials to the environment in the occurrence of those accidents, wherein their influences and consequences are very significant.

Seismic class B: As described above, but the influences and consequences are relatively small.

Seismic class C: Facilities other than those in Seismic class A or Seismic class B, which have only to maintain safety equivalent to general industry facilities.

In short, the concept of this classification is similar to that of Tokai Power Station indicated in Table 7.1, and Seismic class A corresponds to the most important facilities.

In an alternative view, it may be said that, for earthquakes regarded as appropriate from a technological viewpoint to predict their occurrence at the site, the facilities in Seismic class A are those that are potentially capable of causing radiation accidents (when there is a possibility of exceeding 5 mSv per accident) due to the damage of the relevant components, etc. Among the facilities in Seismic class A are those that are potentially capable of inflicting significant radiological hazards far exceeding 5 mSv on the general public in the surrounding area. For such facilities requiring a particular consideration on safety measures, it is believed that seismic design should be conducted in such a way that these facilities can maintain safety functions even against the earthquake ground motion regarded as the upper limit, thereby preventing these facilities from inducing major accidents. The facilities in this class are specifically referred to as those of Seismic class As.

Table 7.2 summarizes what kinds of functions correspond to Seismic class As, A, B, or C.

Here, with regard to facilities relating to so-called “cooling” functions used as safety measures, such as those required for removing decay heat from the reactor core after an accident that causes failure to the reactor coolant pressure boundary, i.e., the emergency core cooling system (ECCS), etc., it is not necessary to categorize them into Seismic class As, and they are categorized into Seismic class A. This is due to the following reason. The objective of seismic design is to inhibit the possibility of accident occurrences even if an earthquake corresponding to either the design basis earthquake ground motion S_1 or S_2 occurred, and prevent major accidents from being induced. Therefore, the design should be conducted in such a way that a loss of coolant accident (LOCA) may not ensue even if an extreme earthquake has occurred. As long as the reactor coolant pressure boundary is intact, even if there happens to be a case where the ECCS may lose its functions as a result of this earthquake, the ECCS is not required. The cooling facilities required to function only needs to remove decay heat from the core after the reactor shutdown (in the case of BWR, the residual heat removal system is designed as Seismic class As). Meanwhile, if an LOCA requiring the ECCS occurred, what would be the levels of earthquake ground motions against which the ECCS should maintain safety functions? The time duration required for cooling after an LOCA is considered to be one year at longest. In that case, the following prediction on the occurrence frequencies is given.

- Occurrence probability of the design basis earthquake ground motion S_1 : approx. 10^{-2} – 10^{-4} /(site year)
- Occurrence probability of LOCA: approx. 10^{-4} /(reactor year).

It could be judged that possible occurrence of the design basis earthquake ground motion S_1 after the occurrence of a LOCA until its safe termination in view of cooling needs to be postulated in seismic design, while similar assumption is not necessary for the design basis earthquake ground motion S_2 (See Ref. [6], p. 31). Therefore, the facility that is required for removing decay heat from the reactor core after an accident of the reactor coolant pressure boundary failure may be categorized into Seismic class A.

Table 7.2 Classification of importance in seismic design in the “Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities, 1981 edition”

Degree of radiation impact on the environment		Classification	Functional classification of nuclear reactor facilities
Facilities containing radioactive materials in themselves or facilities directly related to those containing them, potentially diffusing the radioactive materials to the environment if their functions are lost, and also facilities required to prevent the occurrence of those events and those required to mitigate consequences resulting from the diffusion of radioactive materials in the occurrence of those accidents, and also whose influences are very significant	Among Class A facilities, those particularly important from the viewpoint of safety functions (cooling, stopping, and containing) - Facilities that may potentially cause a loss of coolant accident due to their failure - Facilities required to conduct emergency shutdown of the nuclear reactor, and also to preserve a safe shutdown mode - Facilities for storing spent fuel - Reactor containment vessel	As	i) Components and piping systems composing of the “reactor coolant pressure boundary” ii) Facilities for storing spent fuel iii) Facilities to add negative reactivity for emergency shutdown of the nuclear reactor, and facilities to preserve the shutdown mode of the nuclear reactor iv) Facilities to remove decay heat from the reactor core after the shutdown of the nuclear reactor v) Facilities to prevent the diffusion of radioactive materials directly as the pressure barrier at the failure accident of the reactor coolant pressure boundary
		A	i) Facilities to remove residual heat from the reactor core after the failure accident of the reactor coolant pressure boundary ii) Facilities other than those in v) of Class As mentioned above to mitigate the diffusion of radioactive materials to the environment at an accident which involves the diffusion of radioactive materials iii) Others
As described above, but the influences and consequences are relatively small		B	i) Facilities directly connected to the nuclear reactor coolant pressure boundary and containing the primary coolant or having the possibility to contain the primary coolant ii) Facilities storing radioactive waste, excluding those facilities wherein the consequences of radiation resulting from their failure to the general public are sufficiently small on account of its small inventory or of the type of storage system, as compared with the annual exposure limit at the outside of peripheral observation area. iii) Facilities related to radioactive materials other than radioactive waste, which may potentially cause, as a result of their failure, excessive radiological exposure to the general public and the operational personnel iv) Facilities to cool spent fuels v) Facilities other than those in Class A to mitigate the diffusion of radioactive materials to the environment at an accident which involves the diffusion of radioactive materials
Facilities other than those in Class A or B that have only to ensure safety equivalent to that for general industry facilities		C	Facilities other than those in the abovementioned Class A or B

In the meantime, as described in the safety design regulatory guide, it was stipulated that the importance in the seismic design should be categorized also considering the importance of safety functions. It may appear that this may be correlated with the importance of safety functions. However, since there are matters specific to seismic design, the homologization would have to be conducted cautiously.

(2) **Classification of equipment and supporting structures in facilities [6]**

Although the classification of importance in the seismic design was discussed above in the unit of “facilities,” each facility consists of many constituents. The functions of a “facility” can be achieved only if each constituent fulfills required functions reasonably and systematically. The functions each constituent should fulfill in order to attain the functions required for each “facility” in terms of seismic design vary depending on what role each constituent is playing in the “facility.” Furthermore, detailed design is conducted in the unit of constituents, i.e., components. Therefore, it is necessary to grasp the required functions in unit of constituents.

From such a viewpoint, the constituents are classified focusing on the roles they play in each facility.

- (a) Main equipment: equipment directly related to the relevant functions
- (b) Auxiliary equipment: equipment indirectly related to the relevant functions and plays a role auxiliary to main equipment
- (c) Direct supports: supports directly attached to main and auxiliary equipment, or supports to directly bear loads from the equipment. However, anchor bolts, anchor plates, and embedded plates are classified into this category
- (d) Indirect supports: structures (buildings, constructions), such as reinforced concrete and steel frame structures, etc. subject to loads transmitted from direct supports
- (e) Equipment requiring consideration on mutual influences between themselves: equipment wherein failure of that belonging to a lower seismic class may potentially exert spill-over effects on that belonging to a higher seismic class.

And the seismic design requirements for each of those are as follows.

- Main equipment, auxiliary equipment and direct supports that belong to the same functional classification, from the viewpoint of safety requirements, should be categorized into the same classification of importance in the seismic design and they should meet the seismic design provisions applied to that classification.
- With regard to the evaluation of the supporting functions of indirect supports and the mutual influence between pieces of equipment, it should be confirmed that there are no safety problems against the earthquake ground motions applied to the seismic design of the related equipment. Hence, for example, with regard to the indirect supports subject to the loads transmitted from direct supports, such as ferroconcrete, steel frame buildings and structures, although they do not possess safety functions by themselves, it is required that they do not impede the safety functions of the components they are supporting. In addition, even for buildings, appropriate importance classification should be conducted in cases where

they possess safety functions, such as reprocessing facility buildings possessing a confinement function as well as a supporting function, and biological shields possessing a shielding function.

[2] Design Basis Earthquake Ground Motions

For conducting seismic design, it is necessary to appropriately determine the shaking caused by earthquakes, i.e., the earthquake ground motion of the site ground. The earthquake ground motions used for this seismic design are called the design basis earthquake ground motions. As will be discussed in Sect. 7.3.3, an earthquake occurs through generation of shear fracture along a mechanically weak fault plane as a result of the relative motion of plates. As a result, seismic waves are emitted, and they propagate either in the interior or on the surface of the Earth and shake the site ground. Therefore, in order to determine the earthquake ground motions properly, it is necessary to determine the following items: what kind of earthquake occurrences should be considered first, how seismic waves are generated, how seismic waves propagate, and considering that these waves consequently reach the site and shake the ground and then buildings and components, etc., how the properties of the site ground etc. should be characterized in that case? Now, the concept of determining the design basis earthquake ground motions will be discussed below, focusing on what kinds of earthquake occurrences should be considered.

Concept for Determining the Design Basis Earthquake Ground Motions

In the 1981 Seismic Design Review Guide, the basic objective was defined in accordance with the 1st item of the guideline 2 in the Regulatory Guide for Reviewing Safety Design, and as noted in the beginning of this chapter, it was stipulated that, for the purpose of seismic design, the following performance requirements should be fulfilled (see Ref. [7]).

- (i) Among the earthquakes considered from historical information to have exerted impacts on the site or its surrounding areas in the past and that may occur again and potentially exert similar impacts on the site and surrounding area as well as that may be caused by active faults with high activity and potentially exert impacts on the site in the near future, the most severe earthquake (which induces the largest vibration to the base stratum of the site) shall be selected as the strongest design earthquake. Then, the earthquake ground motion due to this earthquake shall be the design basis earthquake ground motion S_1 . The design shall be conducted in such a way that “facilities and equipment with safety functions of high importance” may remain grossly within the elastic range against the stresses developed by the combination of the seismic force due to this motion and the loads during operation or accidents. Here, the design basis earthquake ground motion means the hypothetical earthquake ground motion on the free surface of the base stratum as the design basis (The definition will be given in Sect. 7.3.3, [6]).
- (ii) On the basis of the situation of earthquake occurrences in the past, the characteristics of active faults in the surrounding area of the site, and seismotectonics,

the study on the engineering aspect of the earthquakes exceeding the strongest design earthquake shall be conducted from a seismological standpoint, and the earthquake ground motion due to the most severe earthquake shall be the design basis earthquake ground motion S_2 . And with regard to the “facilities and equipment with safety functions of high importance, other than those bearing the function of removing decay heat during accidents etc.,” the functions of these facilities shall not be affected by the stresses resulting from the combination of the seismic force due to this motion and the loads during operation or accidents through development of excessive deformation, cracks, failure, etc., even if a considerable portion of the structure undergoes yield and plastic deformation.

These items (i) and (ii) mean that, in order that the general public in the surrounding area or the workers are not affected by radiological hazards even if nuclear reactor facilities encounter large-scale earthquakes, the above-mentioned “facilities and equipment with safety functions of high importance”—i.e., those that would cause radiological hazards if damaged and those for preventing the escalation of radiological hazards—require more stringent seismic design compared to general facilities. And for the earthquakes whose occurrences in the near future are technologically expected, the elastic design should be conducted in such a way that “facilities and equipment with safety functions of high importance” may preserve the state of almost no damage in order that these earthquakes would not induce accidents regarded as the Design Basis Accidents (DBA) in the Regulatory Guide for Reviewing Safety Assessment. These facilities and equipment are those in Seismic class A, including Seismic class As. However, the impacts of the earthquakes exceeding the design basis earthquake S_1 cannot be excluded. Since it is rational to assume upper limit of earthquakes in seismic design from an engineering point of view, the severest earthquake in the area should also be assumed, and required safety functions should be maintained against the earthquake. This allows for designing the facilities in the elastic–plastic range. Although there may also be cases that exceed this ultimate (or critical) earthquake in impact, it was judged that this would be sufficient as engineering countermeasures from the viewpoint of ensuring seismic safety [8]. Facilities and equipment that meet the performance requirements mentioned in (ii) above under the assumption of such critical earthquakes are those in Seismic class As.

Meanwhile, in the “Regulatory Guide for Reviewing Nuclear Reactor Site Evaluation and Application Criteria,” it was stipulated for the subject facilities that, for three states with different occurrence possibilities (probabilities) (i.e., accident, major accident, and hypothetical accident), the impact that each state would bring should be restricted to a certain limit or lower [9]. Although the seismic design requirements should be consistent with this “relationship between occurrence possibilities (probabilities) and damage,” the above-mentioned performance requirements were not necessarily well-organized from the viewpoint of the “the relationship between earthquake occurrence possibility (probability) and consequences” [10].

Earthquakes that should be taken into consideration in order to determine the strongest design earthquake and the critical design earthquake were specifically stipulated as follows.

- (i) Strongest design basis earthquake that causes the design basis earthquake ground motion S_1

Although this is determined from the situation of earthquake occurrences in the past with historical records, it is stipulated that consideration should also be required on the earthquakes resulting from active faults with high activity that may potentially exert impacts on the site in the near future. This consideration is required because there may be inadequacy in old earthquake data, and also it is also necessary to take into consideration the earthquakes resulting from active faults with high activity, so that the occurrence of the earthquakes that have not occurred incidentally over the course of recorded history or those with long repetition intervals may not be overlooked.

With regard to the earthquakes considered to occur due to active faults, a measure for the criteria of evaluation is indicated. Specifically, the following are considered as the generation sources of the design basis earthquake ground motion S_1 .

- (a) Those that have presumably caused earthquakes in the past, according to historical materials.
- (b) Those pertaining to Class A active faults that have been active from 10,000 years ago or have earthquake recurrence intervals less than 10,000 years.
- (c) Those in which the current activities of active faults have been clearly confirmed through the observation of microearthquakes.

Here, Class A activity refers to the activity with the average displacement rate S being larger than 1 mm per year. Class C corresponds to the lowest activity where S is less than 0.1 mm per year. Class B lies in between. It can be considered that active faults with high activity, which are taken into consideration as a source of the strongest design basis earthquake, play the role of compensating for earthquakes in the past with historical records [11].

- (ii) Critical design basis earthquake which causes the design basis earthquake ground motion S_2

In view of the research situation on active faults, etc. in those days when the “Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities, the 1981 edition” was instituted, it was decided that seismotectonics and near-site earthquakes (or, so-called earthquakes directly underneath the site) as well as active faults should be taken into consideration for determining the critical design basis earthquake ground motion. From the viewpoint of seismotectonics and the situation of earthquake occurrences in the past, it can be regarded that there exists an upper limit of earthquake in each earthquake occurrence zone.

Specifically, the following should be considered as generation sources of the design basis earthquake ground motion S_2 .

- (a) Those pertaining to Class A active faults, except those in (b), as sources of S_1
- (b) Those pertaining to Class B and C active faults that were activated during the past 50,000 years or those with earthquake recurrence intervals less than 50,000 years.

As sources of the critical design basis earthquake, seismotectonics and the near-site earthquakes with magnitude 6.5 are taken into consideration in addition to active faults. Seismotectonics should be considered to compensate for the insufficiency of the research on active faults in those days. The necessity for considering near-site earthquakes apparently come from the experience regarding earthquake faults that fault displacement is not generally observed on the ground surface [11].

Since the design basis earthquake ground motion postulated on the free surface of the base stratum can be appropriately expressed by using the maximum amplitude of earthquake ground motion, frequency characteristics of earthquake ground motion, as well as the repetition time and amplitude envelope of earthquake ground motion, these three factors should be obtained from the data of the strongest design basis earthquake and the critical design basis earthquake for determining the design basis earthquake ground motion.

Specifically, at first, the maximum amplitude should be expressed in principle by the maximum velocity among three maximum amplitudes, i.e., maximum acceleration, maximum velocity, and maximum displacement. However, care should be exercised since the acceleration amplitude generally becomes large in the short period domain and tends to affect greatly the design of buildings and structures as well as components and piping. The maximum velocity amplitude in the horizontal direction of the earthquake ground motion on the free surface of the base stratum can be determined by reference to an empirical equation based on the observation results of earthquake ground motions, or theoretical values based on appropriate fault models. Kanai's formula [12] indicated below represents the empirical formula based on observation results. Here, the velocity v_0 [kine] is expressed as follows.

$$v_0 = 10^{f_1} \quad (7.1)$$

$$f_1 = 0.61M - \left(1.66 + \frac{3.60}{X}\right) \log X - \left(0.631 + \frac{1.83}{X}\right)$$

Here, X [km] is the seismic center distance, and M represents the magnitude. The unit [kine] means [cm/s].

Furthermore, the acceleration a_0 [Gal] and the displacement d_0 [cm] on the base stratum are expressed by the following equations.

$$a_0 = \frac{1}{T} \times 10^{f_2} \quad (7.2)$$

$$f_2 = 0.61M - \left(1.66 + \frac{3.60}{X}\right) \log X + \left(0.167 - \frac{1.83}{X}\right)$$

$$d_0 = T \times 10^{f_3} \quad (7.3)$$

$$f_3 = 0.61M - \left(1.66 + \frac{3.60}{X}\right) \log X - \left(1.430 + \frac{1.83}{X}\right)$$

Here, T [s] represents the period of earthquake ground motion, and [Gal] means [cm/s²].

Next, the frequency characteristics of earthquake ground motion is governed by the magnitude of an earthquake, the distance from the center of energy release, and the oscillation characteristics of the base stratum, etc. The so-called Ohsaki's spectrum method [13–15] proposed by Ohsaki describes the frequency characteristics conservatively and had been used for seismic design. The Ohsaki's spectrum was proposed for treating rigid structures installed on the base stratum called the free rock surface. The earthquake ground motions determined in this way are referred to as the design basis earthquake ground motions.

The duration of earthquake ground motions and the time variation of amplitude envelopes are affected by various factors. For example, the duration is affected by the time required for the fracture of faults, etc. Since the duration and time variation are closely correlate with the magnitude, they are determined as a function of magnitude. Specifically, Ohsaki's method [16] is used.

[3] Load Combinations and Allowable Limits

In the “Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities, 1981 edition,” the basic concept on load combinations and allowable limits pertaining to seismic design, which should be investigated in evaluating the adequacy of the design principle for seismic safety, shall be as described below. Moreover, commentaries attached to the Regulatory Guide will also be given in the following text.

1. Buildings and Structures

(1) Seismic class As buildings and structures

- (i) Combinations with the design basis earthquake ground motion S_1 etc. and allowable limits

The load always acting (or normal load) and that act during operation (or operating load) on a facility are combined with either the seismic force due to the design basis earthquake ground motion S_1 or static seismic force, and for the stresses resulting from the combination, the allowable limit should be the allowable unit stress in accordance with the codes and standards that can be recognized as appropriate from the viewpoint of safety.

- (ii) Combinations with the design basis earthquake ground motion S_2 and allowable limits

For the combination of the normal load and the operating load on a facility with the seismic force due to the design basis earthquake ground motion S_2 , the relevant building or structure should have a sufficient margin in deformation capability (toughness) as a structure as a whole, and it should possess an adequate safety margin against the ultimate bearing capacity (or ultimate strength) of the building or structure.

Commentary: The ultimate state of a structure is assumed as the state in which the deformation or stress of the structure has increased significantly by a gradual

increase of the load acting on the structure, and the ultimate bearing capacity means the maximum bearing capacity at the limit of reaching this state.

- (2) Seismic class A buildings and structures (except for Seismic class As)

“Combinations with the design basis earthquake ground motion S_1 etc. and allowable limits” described in (1) (i) above should be applied.
- (3) Seismic classes B, C buildings and structures

The normal load and the operating load on a facility are combined with the static seismic force, and for the stresses resulting from the combination, the allowable limit should be the allowable unit stress described in (1) (i) above.

2. Components and Piping Systems

- (1) Seismic class As components and piping
 - (i) Combinations with the design basis earthquake ground motion S_1 etc. and allowable limits

The loads generated during normal operation, anticipated operational occurrence, and accidents are combined with either the seismic force due to the design basis earthquake ground motion S_1 or static seismic force, and the allowable limits for the stresses resulting from the combination should be the yield stress or the stress with safety equivalent to this.

Commentary: With regard to the “the loads generated during anticipated operational occurrence and during accidents,” it is sufficient for both of them to consider the loads generated by events that may potentially be caused by an earthquake.

In addition, even for the “loads generated during accidents,” if the occurrence of an accident event is very rare and the accident event ends within a very short time period, it is not necessary to consider the load generated by such an event in combination with the seismic force.

In spite of the above, in the case of an event that can last long once it occurs even if it is not possibly caused by an earthquake, the load generated by such an event must be taken into consideration in combination with the seismic force.

- (ii) Combinations with the design basis earthquake ground motion S_2 and allowable limits

The loads generated during normal operation, anticipated operational occurrence, and accidents are combined with the seismic force due to the design basis earthquake ground motion S_2 , and for the stresses resulting from the combination, the functions of a facility should not be affected by the occurrence of excessive deformation, cracks, failure, etc. even in cases where considerable portions of the structure undergo yield and plastic deformation.

- (2) Seismic class A components and piping (except for Seismic class As)

The above mentioned “(1) (i) Combinations with the design basis earthquake ground motion S_1 etc. and allowable limits” shall be applied.
- (3) Seismic classes B, C components and piping

Loads during normal operation and anticipated operational occurrence are combined with the static seismic force, and for the stresses resulting from the combination, the allowable limit should be the yield stress or the stress with safety equivalent to this.

7.3 “Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities, 2006 Edition” [17]

After the institution of the 1981 Seismic Design Review Guide, Japan has experienced strong earthquakes, such as the Southern Hyogo Prefecture Earthquake, and new knowledge has been accumulated on seismology and earthquake engineering, such as types of fault activity and earthquake ground motion characteristics. Consequently, due to the difficulty of classifying seismic sources by relating them to the possibilities of earthquake occurrences, etc., it may not have a rational meaning to set two types of design basis earthquake ground motions, as in the above Guide, and it became desirable to unify them into one. In addition, although it is believed that there would be maximum limits in the strength of earthquakes as mechanical phenomena, the uncertainty pertaining to earthquake phenomena and to the behavior of structures recognized, and the possibility was pointed out that the earthquake ground motions exceeding the unified design basis earthquake ground motion would occur. It was recognized, due to this possibility, that there exist so-called “residual risks” in the seismic design of nuclear reactor facilities.

Consequently, in order to reflect the latest knowledge, etc., to the national guidelines concerning seismic safety, for example, enhancing the reliability of engineering judgment on what had been stipulated in accordance with the engineering judgment in the above Guide, and to make the guidelines more appropriate, the Nuclear Safety Commission of Japan started an investigation in 2001 after preliminary surveys. After the five-year period of the investigation, the Nuclear Safety Commission of Japan instituted a revised version in September, 2006.

The description below will be the background and concept of the revision with regard to the following items that constitute the framework of seismic design, with a focus placed on how the 2006 Seismic Design Review Guide differs from the above-mentioned 1981 Guide.

- (1) Basic policy
- (2) Classification of importance in seismic design
- (3) Method to determine the design basis earthquake ground motions
- (4) Principle of seismic design
- (5) Load combinations and allowable limits.

Figure 7.2 shows, for each of these items, what are the procedures in which seismic design is conducted in accordance with the 2006 Seismic Design Review Guide. This figure focuses on components and piping systems.

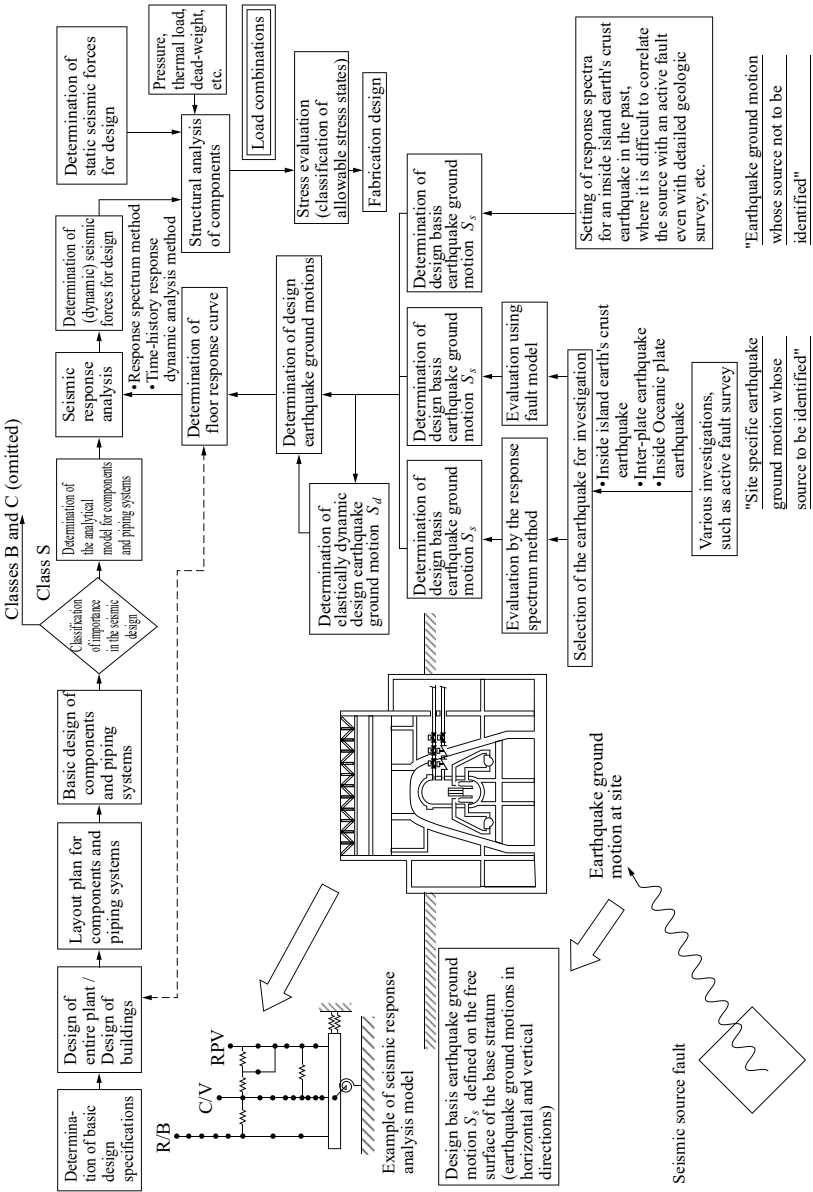


Fig. 7.2 Flow of seismic design (in the case of earthquake-proof Class S components and piping systems)

7.3.1 *Basic Policy*

In revising the 1981 Seismic Design Review Guide reflecting the latest knowledge, the ambiguity arising from abstract expressions was pointed out regarding the 1981 Guide. Hence, the investigation was conducted tracing back to the role of the seismic safety assurance in the safety assurance of nuclear reactors, as well as the purpose and target of seismic design.

Seismic design is, in a word, specifying the targets as to what kinds of earthquakes and in what states that buildings and components of nuclear power reactor facilities, etc. must withstand, and designing so as to fulfill the targets. While an earthquake sways all the buildings and components of nuclear reactor facilities as a common initiating event, the hazard potential pertaining to the radioactive materials stored in each building, component, etc. differs by the building, component, etc. Similarly to the concept of conventional seismic design, it is rational to assume the earthquake magnitude according to the importance of buildings, components, etc. Since it is possible to think of various design methods with which each building, component, etc. can meet the above-mentioned targets, it would be better to select appropriate design methods considering the current technological level.

Firstly, with regard to the target of seismic design, in principle, earthquake levels that would not cause radiological hazards to the general public in the surrounding area are specified as the design basis earthquake ground motions. However, it is necessary to recognize, from the previous knowledge, that there exist earthquake ground motions that exceed these design basis earthquake ground motions. Therefore, in the course of the revision, consideration of the following two items as targets was basically pursued [10]. Among these items, although target II was not explicitly stipulated in the eventual Basic Policy, what is meant by this item is that “residual risks” exist in the design of nuclear reactor facilities.

Target I: It is a matter of course that the design, construction, operation, and maintenance should be conducted in such a way that nuclear reactor facilities may not cause accidents, even if they experience earthquake ground motions that would occur more than once during the design life, in view of the characteristics of the area surrounding the site. Furthermore, the earthquake ground motions that may seem to occur during the life of the facility, from the viewpoint of seismology on the basis of the relationship between the magnitude and frequency of earthquake ground motions determined by the situations in the area surrounding the site, shall be defined as the design basis earthquake ground motions, and the design shall be conducted in such a way that, even if these motions are assumed to occur, the essential safety functions including those of safety protection facilities may not be damaged, and the general public in the surrounding area may not be subject to radiation disasters.

Target II: Even considering the possible occurrence of earthquake ground motions exceeding these design basis earthquake ground motions, the resulting risks of radiation disasters to the general public shall be small, owing to the design margins for facilities.

The concept of the design basis earthquake ground motions defined in target I has been enhanced in such a way that the previous knowledge could be reflected, and also the issues pointed out in the 1981 Seismic Design Review Guide could be addressed.

In the concept of the design basis earthquake ground motions in the 1981 Guide above, two types of earthquake ground motions, S_1 and S_2 , had to be determined as noted in Sect. 7.2.3. The Earthquake causing the design basis earthquake ground motion S_1 (strongest design basis earthquake) is an earthquake for which it is appropriate to anticipate its occurrence from the engineering viewpoint. Since there are cases where the occurrence of earthquakes exceeding the strongest design earthquake cannot be denied, the earthquake causing the design basis earthquake ground motion S_2 (critical design basis earthquake) is an earthquake that should be investigated from the engineering viewpoint and is postulated as an earthquake whose occurrence in future cannot be denied. In determining these earthquake ground motions, the earthquakes that would conceivably occur due to active faults were considered by classifying them in terms of the activity of the faults.

However, it had been pointed out that the activity levels of active faults themselves were not much related to the occurrence probability of earthquakes in the near future, and it would not be appropriate to classify earthquakes directly by fault activity. Therefore, in light of accumulated knowledge in the latest seismology and earthquake engineering, etc., it has been decided as the basis of the seismic safety requirements that the design basis earthquake ground motions shall be unified into one type, and the safety functions of the facilities that are important from the viewpoint of seismic safety shall be maintained against seismic forces due to the design basis earthquake ground motion S_s . However, it is in the state of elastic limit that the impacts that earthquakes exert on facilities and the states of facilities can be evaluated by applying the current design technology. And therefore, in order to confirm the safety functions of facilities against the design basis earthquake ground motion S_s with higher accuracy, it was proposed, in the course of the investigation for revision, that the elastically dynamic design earthquake ground motion shall be set from the engineering viewpoint, and it has been decided that the elastically dynamic design earthquake ground motion S_d shall be set as S_s multiplied by a certain factor.

In addition, with regard to the design methods, although it was stipulated in the 1981 Seismic Design Review Guide that “buildings and structures shall be rigid structures in principle,” and “important buildings and structures shall be supported on the base stratum,” in view of maturity of the latest technology such as base isolation, these descriptions have been deleted from the reasons given below and upgraded to “shall be settled on the ground that has sufficient supporting capacity” [18].

Regarding the description “buildings and structures shall be rigid structures in principle”:

Although the reason for limiting to rigid structures had been to avoid resonance with earthquakes, etc., design technology regarding base isolation structures, etc. other than rigid structures has made dramatic progress, and actual cases of applying base isolation structures to nuclear facilities are on the rise particularly in foreign countries. In view of these situations, it was judged that the seismic safety either equal to or surpassing that for rigid

structures could be ensured, even with the design methods of buildings and structures other than those for rigid structures, and therefore it was decided that this description should be deleted.

Regarding the description “important buildings and structures shall be supported on the base stratum”:

“Supported on the base stratum” had been required mainly because of the following reasons.

- Since the base stratum has sufficiently high strength, it is suitable for supporting weighty buildings such as important buildings and structures of nuclear reactor facilities which would generate large seismic forces.
- In the case of siting on the base stratum compared with the subsurface layer, the amplification of shaking from the seismic source can be made smaller.

In view of the progress in design technology, such as Quaternary deposit siting, it was judged that, if it can be demonstrated that buildings and structures are settled on the ground possessing sufficient capability to support them in accordance with the design loads, sufficient seismic safety can be ensured even if they are not supported on the base stratum, and the description has been revised to “buildings and structures shall be settled on the ground with sufficient ability to support them.”

Through such investigations, the basic policy has been revised as follows.

“Facilities important from the viewpoint of seismic design shall be designed in such a way that the safety functions may not be damaged against seismic forces resulting from earthquake ground motions, which have the possibility of occurrence during the operational period of the facility in view of seismology and earthquake engineering regarding the geology, geologic structure and seismic activity of the area surrounding the site although it may be quite scarce, and for which it is appropriate to postulate the possibility of exerting significant impacts on facilities.

In addition, buildings and structures shall be settled on the ground that has sufficient capability to support them.”

7.3.2 Importance Classification in Seismic Design

In the 1981 Seismic Design Review Guide, the importance in the seismic design was classified into Classes A, B, and C, and particularly important facilities in Class A from the viewpoint of safety measures were classified into Seismic class As. Mainly from the following three reasons with regard to this classification, Seismic classes A and As have been unified noting that the necessity for this demarcation was not obvious. Regarding the names, after twists and turns, As and A have been unified into a new Seismic class called S, while Seismic classes B and C remained as they had been in the 1981 Seismic Design Review Guide.

- (1) In the seismic design in accordance with the 1981 Seismic Design Review Guide, the difference in importance (compared in terms of the frequency of reactor core damage) was not large between Seismic classes As and A according to the

results of the seismic probabilistic risk assessment (PRA), where the impact of earthquakes was evaluated probabilistically including large-scale earthquakes exceeding the critical design basis earthquake. The reason behind this is that, in the actual design, the components in Seismic class As and those in Seismic class A may be designed in such a way that their seismic safety margins do not differ significantly [19].

- (2) The design basis earthquake ground motions have been generalized to S_s .
- (3) Correlation with the classification of importance of safety function has been taken into consideration.

With regard to (1), the impacts due to large-scale earthquakes exceeding the critical design basis earthquake had been primarily considered in the 1981 Seismic Design Review Guide as follows [8]. Namely, So long as the required safety functions (or safety measures) were ensured against the critical design basis earthquake as well as the strongest design basis earthquake, it would be sufficient as an engineering action even if there may be cases where the critical design basis earthquake would be exceeded. Here, the safety functions that should be ensured also during the critical design basis earthquake were, as shown in the Seismic class As column in Table 7.2, the safety functions pertaining to the integrity of reactor coolant pressure boundary, the shutdown of reactor and the maintenance of the shutdown state; the integrity of reactor containment boundary, and the safety functions of facilities for storing spent fuel, all of which were minimum requirements for ensuring the three major functions of “confining,” “cooling” and “stopping,” whereas the latter two were indispensable for the former. Furthermore, the design should allow for plastic deformation if the required functions were ensured. Meanwhile, during the strongest design basis earthquake that would occur more frequently than the critical design earthquake, it was necessary to ensure the following functions in addition to the safety functions of the aforementioned facilities, for reasons indicated in the example described in Item [1] (1) in Sect. 7.2.3, etc. Namely, the safety functions of the other engineered safety features like the ECCS, and the safety functions of facilities for storing substances that could potentially diffuse a large amount of radioactive materials, although not directly related to the core damage. In addition, against the strongest design basis earthquake, it was required from the viewpoint of restart, etc. that the assurance of these safety functions should be attained in the elastic design. For these reasons, among the facilities in Seismic class A, the former facilities were classified into Seismic class As in the classification of importance in the seismic design. This means, in other words, if the safety functions of facilities in Seismic class As were ensured even during the critical design basis earthquake, it was not necessary to design facilities in Seismic class A in such a way that they could withstand the critical design earthquake. Given that the classification of the functions required for facilities in Seismic classes As and A is as described above, where did such a difference come from? As it will become clear if the example in Item [1] (1) in Sect. 7.2.3 is scrutinized, this is because the difference resides in the period of time during which the safety functions have to be functional after the occurrence of an accident. The safety functions possessed by the Seismic class A facilities would

become unnecessary to function in relatively early stages, perhaps in a year or so, after the occurrence of an accident. In contrast, the reactor containment boundary, etc. were required to be functional over a long period of time.

In recent years, the seismic PRA has started to be conducted as part of risk assessment, and the following findings have been obtained [19]. In the seismic PRA, a damage probability under earthquake ground motions is obtained for each facility and component, and the core damage frequency (CDF) is derived from this result through reliability analysis. In doing so, importance becomes clear, i.e., what kind of and how facilities and components are significant from the viewpoint of the CDF. If we also consider an earthquake exceeding the critical design basis earthquake, and in particular, suppose a large-scale earthquake that may even break facilities of Seismic class As, the importance of facilities in Seismic class As will not be different from the case of Seismic class A. Thus, in consideration of major earthquakes exceeding the critical design earthquake, it has become clear that facilities in Seismic class A gained importance further, and there was no difference in importance between Seismic classes As and A. To put it another way, this means that, although there is a difference between facilities in Seismic classes As and A in terms of the period of time during which the safety functions should be functioning after the occurrence of an accident as described above, it is difficult to find out the quantitative difference in values that contribute to the reduction of risks pertaining to the diffusion of radioactive materials [20]. For this reason, it was pointed out that As and A be unified.

With regard to (3), the classification of importance in the seismic design is primarily stipulated in Guideline 2 of the Regulatory Guide for Reviewing Safety Design [21]. More specifically, it is stipulated that structures, systems and components possessing safety functions shall be classified from the viewpoint of seismic design, considering the importance of safety functions and also the influence on safety when their functions are lost due to earthquakes. Among other guides that stipulate the classification of importance pertaining to safety functions for the basic design is the “Regulatory Guide for Reviewing Classification of Importance of Safety Function of Light Water Nuclear Power Reactor facilities” [22] (hereafter referred to as the “Regulatory Guide for Reviewing Classification of Importance of Safety Function”). This regulatory guide was triggered by the Three Mile Island-2 accident and instituted considerably later than the 1981 Seismic Design Review Guide. The classification was prescribed in consideration of the influences that structures, systems and components (SSC) would exert on safety when their functions are lost. More specifically, they are at first broadly classified into two groups based on the nature of safety functions, and then the constituents of each group are classified into three Classes 1, 2, and 3, according to the importance. The two groups consist of the SSC that may potentially cause accidents, etc. when their functions are lost (abnormality prevention systems, or PS), and the SSC that aim for the mitigation of impact, e.g., the prevention of escalation when accidents occur (abnormality mitigation systems or MS). The importance classifications in both guides are conducted in consideration of the influence on safety when safety functions are lost, and it is natural that they have many points that are logically common.

However, since there are items specific to seismic design, e.g. the pursuit of prevention of secondary accidents during earthquakes, and the consideration on the possibility of simultaneous damage to components, etc. during earthquakes [23], it is not appropriate to apply the classification of importance of safety function as it is.

Mainly for the aforementioned reasons, the importance in the seismic design shall be classified into three classes. And reflecting the comments made during the public review for the revision of the guide, the three classes shall be referred to as S, B, and C, respectively. The safety functions possessed by facilities in each Seismic class are shown in Table 7.3.

7.3.3 Method for Determining the Design Basis Earthquake Ground Motions

The concept of determining the design basis earthquake ground motions has advanced significantly from the 1981 Seismic Design Review Guide, reflecting the knowledge of the latest seismology, etc. Hence, before going into the main subjects, let's summarize the current understanding as to how the earthquakes and the method for computing the propagation of seismic waves from the source to a site are being considered. Here, the design basis earthquake ground motions shall refer to the earthquake ground motions used for the seismic design of facilities in the highest class of importance classification in seismic design, and these motions shall be defined on the free surface of the base stratum.

[1] About Earthquakes

(1) Mechanisms of Earthquake Occurrence

An earthquake occurs as a result of abrupt slippage, on a weak plane called a fault plane within a rock mass or a plate boundary where crustal masses called plates collide with each other. The slippage in the fault plane arises because when the strain acting on a rock mass concentrates on this weak plane, the shear force that tends to cause slippage of the rock mass concentrates there and eventually exceeds the shear fracture strength and causes shear fracture, finally leading to rupture. Shear fracture strength is essentially static friction force. Being in a rock mass, the fracture is brittle. Among the fracture modes are both in-plane brittle mode II (edge sliding mode) and out-of-plane brittle mode III (shear mode) in the classification in fracture mechanics, as illustrated in Fig. 10.2 of Sect. 10.2. A fault that has generated an earthquake, i.e., that has slipped and caused a dislocation, is referred to as a seismic source fault. A seismic source fault has an expanse and its entire plane does not necessarily slip concurrently with the initiation of the earthquake.

Just like a dislocation movement generating plastic deformation, the slippage starts at one point and then spreads over the plane. Therefore, strictly speaking, the source is the location on a seismic source fault where the slippage has initially started,

Table 7.3 Classification of importance in seismic design in the “Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities, 2006 edition”

Degree of radiation impact on the environment	Classification	Facilities in each class
Facilities containing radioactive materials in themselves or related directly to those containing them, potentially diffusing radioactive materials to the environment if their functions are lost and facilities required to prevent the occurrence of those events, and those required to mitigate the consequences resulting from the diffusion of radioactive materials in the occurrence of those accidents, and also whose influences are very significant	S	i) Components and piping systems composing of the “reactor coolant pressure boundary” ii) Facilities for storing spent fuel iii) Facilities to add negative reactivity for emergency shutdown of the nuclear reactor, and facilities to preserve the shutdown mode of the nuclear reactor iv) Facilities to remove decay heat from the reactor core after the shutdown of the nuclear reactor v) Facilities to remove residual heat from the reactor core after failure accident of the reactor coolant pressure boundary vi) Facilities to prevent the diffusion of radioactive materials directly as the pressure barrier at the failure accident of the reactor coolant pressure boundary vii) Facilities other than those in mentioned above to mitigate the diffusion of radioactive materials to the environment at the accident, which involves the diffusion of radioactive materials
As described above, but the influences are relatively small	B	i) Facilities directly connected to the nuclear reactor coolant pressure boundary and containing the primary coolant or having a possibility to contain the primary coolant ii) Facilities storing radioactive waste, excluding those facilities wherein the consequences of radiation resulting from their failure to the general public are sufficiently small on account of its small inventory or of the type of storage system, as compared with the annual exposure limit at the outside of the peripheral observation area iii) Facilities related to radioactive materials other than radioactive waste, which may potentially cause, as a result of their failure, excessive radiological exposure to the general public and the operational personnel iv) Facilities to cool spent fuels v) Facilities other than those in Class S to mitigate the diffusion of radioactive materials to the environment at an accident which involves diffusion of radioactive materials
Facilities other than those in Class S or B that have only to ensure safety equivalent to general industry facilities	C	Facilities other than those in the abovementioned Class S or B

and more specifically, it is defined as the location where the shear fracture has started. The distribution of this slippage is not uniform on the seismic source fault plane, and there are places where the slippage is large and not. This can be understood if the mechanism of shear fracture is scrutinized more in detail. Conceptually, rock masses are not perfectly sticking together in a fault plane, but the plane is undulated. Although they are pressing each other strongly at protruded areas to prevent slippage, if these areas exceed the shear fracture strength and fracture, a large slippage will arise in the pressed areas. A region with large slip, where was strongly stuck and causes a large slippage when it occurs, is referred to as an asperity. A ground displacement extending to the earth's surface is referred to as a surface earthquake fault.

If a shear fracture occurs in a fault, it shakes the surrounding rock masses, and the shaking propagates as seismic waves.

(2) Occurrence Locations

Let's summarize while quoting descriptions from Refs. [24, 25]. In and around the Japanese Islands, as schematically illustrated in Fig. 7.3, there are four huge crustal masses called plates, which are pressing each other. Near the Japanese Islands, the oceanic Pacific Plate advances from the east toward the continent side plates and subducts beneath them along the Kuril, Japan, and Izu-Bonin Trenches. It is said that the subducted Pacific Plate extends beyond the Japan Sea and reaches the depth of more than 500 km beneath the Eurasian Continent. Meanwhile, the Philippine Sea Plate advances from the south and subducts beneath the continent side plates along the Nankai Trough and the Ryukyu Trench. It is said that this subducted Philippine Sea Plate extends to the depth of more than 200 km beneath the Kyushu and Okinawa areas. As such, the Pacific Plate is harder to bend than the Philippine Sea Plate, which is conceivably because the Pacific Plate is older and thus has been cooled more and has become thicker than the Philippine Sea Plate.

Earthquakes tend to occur along the plate boundaries where the Pacific and Philippine Sea Plates on the Pacific Ocean side are subducting beneath the continent side Eurasia Plate and the Okhotsk Plates, which may also be considered as part of the North American Plate. Figure 7.4 shows the conceptual diagram. These earthquakes are called inter-plate earthquakes, which are more likely to become strong earthquakes. A trench or trough is formed where subduction occurs. The term "trench" is used for cases where the water depth exceeds 6000 m, while the "trough" is for cases where this depth is not exceeded. The Great Kanto Earthquake (magnitude 7.9) that occurred in September 1923 was an inter-plate earthquake. An oceanic plate subducts beneath a continent side plate, because the former is heavier than the latter as an entire plate. Regarding how an inter-plate earthquake occurs, as the oceanic plate subducts, the continent side plate is dragged by this movement and bent downward. When this bending reaches the limit, it jerks upward to return to the former position. At this moment, a strong earthquake occurs and breaks the plate boundary. Other than this case, when two continent side plates come close to each other, a collision occurs. Subsequently, one plate subducts beneath the other plate. In the meantime, there is an objection [26, 27] to the theory that there are two continent side plates bounded by the Fossa Magna. This objection theorizes that the earthquakes occurring in the

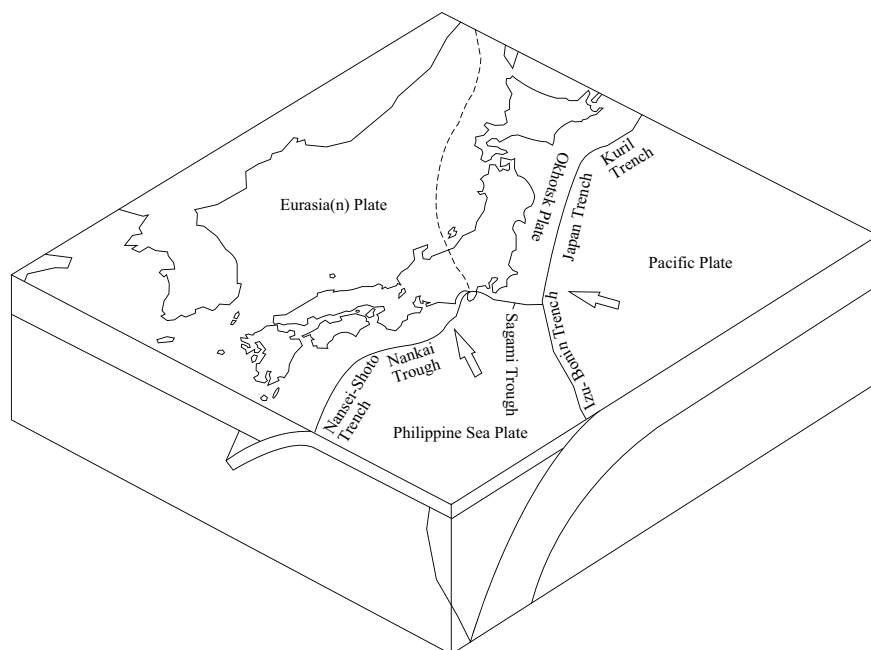


Fig. 7.3 Plates in and around Japan and their movements [24] (The boundary between the Okhotsk and Eurasia Plates is shown by a broken line because its location has not been established completely)

eastern margin area of the Japan Sea should be considered equivalent to those occurring in the in-land fault zones, rather than considering the above-mentioned margin as a plate boundary.

Earthquakes occur not only along plate boundaries, but also within a plate. Those of the latter case are called inside plate earthquakes. Among inside plate earthquakes are earthquakes that occur in shallow places at the depth of up to about 20 km, such as those occurring in the inland of Japan, and earthquakes occurring within either a subducting oceanic plate or a subducted plate. The earthquakes that occur in shallow locations, such as the Southern Hyogo-ken Earthquake in 1995, are caused by the repetitive activity of faults. However, the recurrence period of these repetitions are long compared to that of inter-plate earthquakes, and may extend over tens of thousands of years. Faults that have experienced repetitive activity in the Quaternary, particularly the late Quaternary period (after the last interglacial period 120 thousand years ago), are said to have the possibility of future reactivation.

Besides these, there are volcanic earthquakes. However, since nuclear reactor facilities are not installed near volcanoes, they are excluded from the consideration in this chapter.

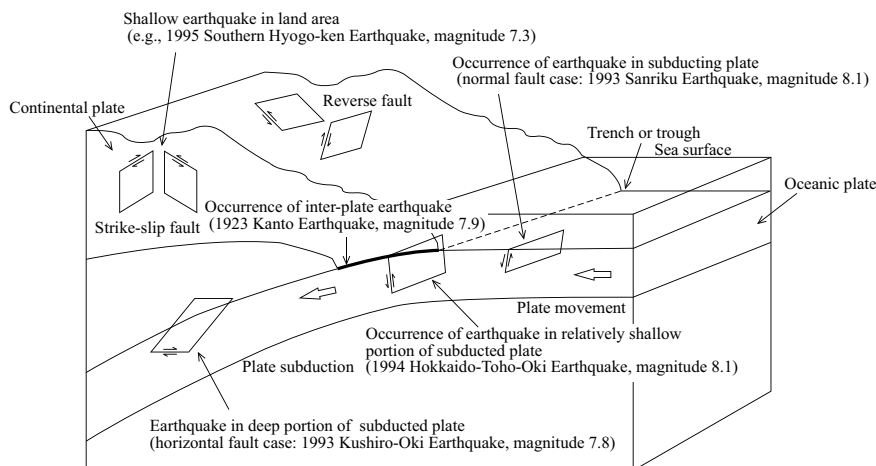


Fig. 7.4 Types of earthquake occurring in and around the Japanese Islands (see Fig. 7.5 for the kinds of fault. The earthquakes in parentheses are examples. This figure was prepared with some modifications to Ref. [24])

[2] On the Computation of Seismic Wave Propagation from Source to Site (Prediction of Earthquake Ground Motions at Site)

Seismic waves emitted by slip fracture (shear fracture) of a fault start from a source and travel to the earth's surface as they undergo reflection and refraction in the ground. Generally, there are two paths for the propagation of seismic waves. One is the case where seismic waves propagate in the ground and directly arrive at an evaluation point. These seismic waves are called body waves and consist of the P-wave, a longitudinal wave oscillating in the same direction as the seismic wave travels, and the S-wave, a transverse wave oscillating in the perpendicular direction. Here, "P" in the P-wave denotes the initial letter of "primary," indicating the wave that arrives first, and "S" in the S-wave denotes "secondary." Another wave is the surface wave that reaches somewhere on the earth's surface other than the evaluation point and then propagates the surface and arrives at the site.

Here, let's introduce definitions that are used as a basis for defining seismic waves in the seismic design in general industry, although it is different from the definition of the free surface of the base stratum. First, the base stratum in which the S-wave propagates at the velocity of about 3 km/s is defined as the "seismic base stratum." In underground structures deeper than the seismic base stratum, the amplitude of a seismic wave gradually attenuates as it travels away from the source, but the velocity change is relatively small, and changes in the wave due to reflection and refraction are also not large. In addition, a geologic layer in which the velocity of the S-wave is about 0.4 km/s is defined as the "engineering base stratum," and the geologic layers above this layer are called the subsurface soil.

Getting back to the subject, the amplitude of seismic waves emitted from a seismic source attenuates as they travel away from the source, and this phenomenon is called

the “attenuation with distance.” In computing the propagation of seismic waves, it is very important to consider how this attenuation with distance should be estimated. One of the causes of the attenuation with distance is that seismic waves spread as they travel away from the source. This is called the “geometric attenuation.” In this case, the amplitude attenuates in inverse proportion to the square root of the distance from the source. In addition to this, there are attenuation due to the viscosity of geologic layers (called the “viscous attenuation”) and that due to scattering arising from heterogeneous underground structures (call the “scattering attenuation”).

Therefore, in order to predict the shaking of earth’s surface due to seismic waves, it is necessary to evaluate the propagation paths of seismic waves from the source to the evaluation point, the effects of the subsurface soil at the evaluation point, let alone to evaluate the properties of the seismic source fault, and predict the shaking at the site by using the computation method that can reflect the above-mentioned information. Namely, it is important to conduct the following items appropriately in order to predict the shaking and earthquake ground motions at the evaluation point.

- (1) Modeling of a source (or seismic source fault) (fault structure, fracture initiation point, fracture propagation mode, etc.)
- (2) Modeling of underground structure (including propagation paths and subsurface soil)
- (3) Selection of the method for computing earthquake ground motions.

Let’s summarize what have been clarified and proposed in the previous researches. Regarding the first item, modeling of a source (or seismic source fault), it has come to be accepted that the shaking of the ground surface when a strong earthquake occurred (this is called an “intense ground motion”) is significantly affected by the heterogeneity of the fault structure (i.e., asperity). Hence, it has been proposed to perform modeling by separating areas with asperity from the other background areas.

Broadly speaking, four methods have been proposed for computing earthquake ground motions that incorporate such a source model as well as subsurface structure model.

- (i) Empirical method
- (ii) Theoretical method
- (iii) Semiempirical method
- (iv) Broadband frequency hybrid method (in which the aforementioned methods are combined for predicting strong ground motions in a wide frequency range).

Let’s describe the features of each method while citing from Ref. [28], and Ref. [29], etc. are recommended for those who intend to study further.

(i) Empirical method

This method aims to express strong ground motions simply with only a few parameters through statistic analysis of a number of records of strong motions. This is often used for predicting strong ground motions in a short period range. A typical example is called a “formula for attenuation with distance,” which statistically analyzes the

observation data and predicts the maximum acceleration, maximum velocity, and response spectra, which are the indices of shaking, from the magnitude of the earthquake and the distance from the source, etc. Originally, a magnitude scale was proposed by C. F. Richter in 1935 as “the logarithm of maximum displacement amplitude, read in microns, of the seismic waves observed at a point 100 km away from the epicenter measured using a type of seismograph called the Wood-Anderson type” for an earthquake that occurred in Southern California, USA [30]. In this way, it had been considered empirically that the intensity of earthquake ground motion could be expressed by the magnitude and distance.

Here, the response spectrum is explained in the following. Among the methods for describing waves generated by earthquakes, or seismic waves, are a method of directly analyzing and describing them as well as a method of describing seismic waves while focusing on the response of the structures in such a way that the impacts exerted by the seismic waves can be easily grasped. The response spectrum corresponds to the latter, in which a seismic wave is expressed as maximal response values so that it is easy to grasp how buildings, components, etc. shake due to the earthquake ground motion, in other words, how they respond. A seismic wave form can be expressed by summing up periodic functions with suitable amplitudes and phases. Meanwhile, since the natural periods of buildings, components, etc. are different, the maximal amplitude value of each component, etc. depends on seismic wave forms. Therefore, various oscillating systems with one degree of freedom that simulate respective components, etc. are postulated and placed on the base stratum. Each oscillating system is set in such a way that it has different natural period from others. If the base stratum is shaken by the seismic wave for evaluation, since the oscillating systems have different natural periods, they also have different maximal response amplitude pertaining to displacement, velocity and acceleration. A response spectrum is the plot of these data, where the horizontal axis represents the natural period and the vertical axis represents the maximal response amplitude of the system of the corresponding natural period. In seismic design, postulated seismic waves are often represented by such response spectra.

(ii) Theoretical method

A physical model of fault movement and underground structure in which seismic waves propagate is developed, and based on the theory of seismic wave generation and propagation, time-history wave forms of earthquake ground motions are computed deterministically. This model is used frequently for simulation of strong ground motions in the band of somewhat longer periods. Among the methods of treating underground structure are theoretical and numerical analysis methods. The former is applied to simple underground structures for now, while the latter is used for complicated cases such as the modeling of subducting plate structures, etc. The latter includes the finite difference and finite element methods.

(iii) Semiempirical method

This method compensates for the theory partially with actual records of small earthquakes and is often used for simulation of strong ground motions involving a band of short periods where probabilistic phenomena (random phenomena) prevail. In regard to this, a predominant period refers to a period at which shaking becomes large when viewed as a function of period. When the predominant period and the natural period of a structure coincide, the structure will resonate. Since a number of earthquake ground motion waveforms pertaining to small earthquakes are superposed in this semiempirical method, it is sometimes called the semi-empirical synthetic seismogram method. This method differs depending on whether or not records are available for small earthquakes at the prediction point of strong ground motions. When the records of strong motions (observed waveforms) due to small earthquakes that has occurred in the same source area as a postulated intense earthquake are available at the prediction point, the time-history waveforms of the postulated earthquake are obtained by taking the fault fracture process of the postulated earthquake into consideration, regarding the waveforms observed in small earthquakes as Green's functions, and superposing them in large numbers. This approach is called the empirical Green's function method. Green's function represents a response at the observation point when a unit force is applied to the source. This method assumes the following two things. One is that fault movement in a fault plane during an intense earthquake can be expressed in accordance with an empirical rule by superposition of a large number of fault movements pertaining to small earthquakes. The other is that the propagation characteristics of seismic waves emitted from a certain point on a fault plane are the same, whether it is an intense or small earthquake, if the observation location does not change. Under these assumptions, earthquake ground motion waveforms due to a small earthquake at the observation point are represented by empirically obtained Green's functions. However, this is difficult to apply when appropriate records of small earthquakes are not available. In addition, this is also difficult to apply directly when nonlinear behavior of the ground, which is not observed in small earthquakes, emerges during intense earthquakes.

Another method that has been contrived for cases where appropriate records of small earthquakes are not available utilizes simulated seismic waves created by statistically processing multiple records of strong motions obtained at different points, while regarding the waves as statistically evaluated Green's functions. This is called the stochastic Green's function method.

(iv) Broadband frequency hybrid method

Three types of computation methods discussed so far have their own merits and demerits. Recently, the range of periods to be predicted for strong ground motions is getting wider, and it is becoming difficult to cover such a wide period band by simply using one of these methods. Hence, the broadband frequency hybrid method aims to obtain strong ground motions in a broadband by dividing the band into long and short period bands, computing the predicted waveforms of strong ground motions

for respective bands by using suitable approaches, and subsequently summing up both results in a complementary manner. Practically, in a long-period band, since deterministic phenomena are predominant in the phenomena in a seismic source fault, the theoretical method is applied, whereas in a short period band, probabilistic phenomena (random phenomena) of fracture prevail, and the semiempirical method is applied.

So much for the preliminaries and let's get down to the main subjects including the unification to the design basis earthquake ground motion S_s , associated "residual risks," earthquakes investigated for determining the design basis earthquake ground motion, and methods for computing the design basis earthquake ground motion due to these earthquakes.

[3] Unification to the Design Basis Earthquake Ground Motion S_s and "Residual Risks"

After the Great Hanshin-Awaji Earthquake Disaster due to the Southern Hyogo Prefecture Earthquake, seismology and earthquake engineering, in particular strong motion seismology and seismic hazard assessment technology, have progressed significantly, and in view of the necessity for reflecting the latest knowledge from these results, the Seismic Design Review Guide was reviewed. In the process of this review, as described above, with regard to the design basis earthquake ground motions, it was pointed out mainly from science viewpoints that there was no rationality in determining the two design basis earthquake ground motions. Consequently, the design basis earthquake ground motions have been unified into S_s , retaining the implication of S_2 .

However, the necessity for setting the earthquake ground motion for elastic design was pointed out from the engineering side, and the elastically dynamic design earthquake ground motion S_d has come to be defined based on S_s . The meaning of this earthquake ground motion is to realize sufficiently reliable seismic design by using elastic design, taking over a portion of the role that S_1 had played in seismic design. Actually, this means that the design shall be conducted in such a way that, even if strong earthquakes corresponding to S_d occurs, a nuclear reactor can be shut down safely and then restarted subsequently.

In determining S_s , as described below, it is stipulated that geologic survey, etc. shall be conducted more minutely and carefully than before, and physical and tectonic relief geomorphologic examinations shall be newly specified, and up-to-date methods shall be applied. It is important to recognize that, even with such leading edge technology, the evaluation of the design basis earthquake ground motions accompanies uncertainty, for example, with regard to the formula for attenuation with distance. From this viewpoint, it is stipulated that "residual risks" shall be recognized and minimized to the lowest possible level. The "residual risks" are defined as the "risks pertaining to the occurrence of serious damaging events in facilities due to the effects of earthquake ground motions impacting thereon that exceed the specified earthquake ground motions, and the occurrence of events in which a large amount of radioactive materials diffuse from facilities, or as a result of these events, causing radiological exposure hazards to the general public in the surrounding area." These residual risks

can be grasped by a probabilistic seismic risk assessment (also call a seismic PRA). Furthermore, it is stipulated that the exceedance probabilities of the response spectra pertaining to the design basis earthquake ground motion S_s shall be estimated and utilized as reference information during the safety review. It is required that these exceedance probabilities shall also be utilized in investigating how far to consider the combination of seismic load and loads during accidents. Here, it is stipulated, as a policy decided by the Nuclear Safety Commission of Japan when the Seismic Design Review Guide was revised, that the assessment of “residual risks” shall be experimentally conducted independently of the current safety regulations for the time being, with its full introduction to future safety regulations in mind [31]. This is because there still are matters to be discussed; for example, an agreement has not been reached even among the experts regarding the introduction of probabilistic approaches for evaluating “residual risks,” and the quantitative target values for safety regulations have not been set as allowable risks to nuclear reactor facilities, etc.

[4] Earthquakes that Shall Be Evaluated in Determining Design Basis Earthquake Ground Motions

First, in determining a design basis earthquake ground motion, recognizing that there exist faults that cannot be grasped even with the current technology of geologic survey, etc., maximum effort shall be exerted to identify the faults that may become a seismic source or seismic source fault, and geologic survey, etc. shall be conducted with utmost minuteness using the latest technology. In this way, earthquake ground motions shall be evaluated on the basis of identified faults on the safe side. Since there still are faults that cannot be grasped, the “earthquake whose source cannot be specified” shall be investigated. This concept is stipulated more clearly than in the 1981 edition, although it already existed in the latter. More specifically, even if detailed examinations are conducted fully considering the situation of the surrounding area of the site, it cannot be concluded that it is possible to evaluate beforehand all the earthquakes having the possibility of occurrence in the vicinity of the site and that may potentially be generated within the crust of the continent side plate. Hence, the “earthquake whose source cannot be specified” refers to the earthquake ground motion that should be commonly taken into consideration irrespective of the results of detailed examinations in the vicinity of the site. In addition, large-scale earthquakes do not necessarily coincide with strong earthquake ground motions at the site. Therefore, it is necessary to configure an earthquake that would cause strong earthquake ground motions, focusing on the earthquake ground motions that each seismic source fault would generate at the site.

In the meantime, if the modes of earthquake occurrence (including the configuration, movement, and interaction of plates) differ in light of plate tectonics, then the characteristics of the source, the regionality of seismic activities, and the characteristics of seismic wave propagation, etc. will be different, and therefore, it is necessary to determine earthquake ground motions by classifying them according to the modes of occurrence. For example, an earthquake occurring in a deep subducted portion of an oceanic plate (although this subducted portion is sometimes referred to as a “slab,” care should be taken because in some cases a subducted plate is referred

to as a slab) differs in the characteristics of the source, seismic wave propagation, etc. in comparing with an inter-plate earthquake occurring along the boundary of a continent side plate and an oceanic plate etc. and an earthquake occurring in a shallow portion of a continent side plate (referred to as an “inside island earth’s crust earthquake” or “inland crustal earthquake”) as explained below. Regarding the characteristics of the source, the ground acceleration becomes large because high frequency components are excited strongly. Regarding the propagation characteristics, the seismic intensity distribution of an earthquake occurring in the Pacific Plate is elongated along the strike of the slab without forming a concentric pattern around the epicenter as observed in other cases, and it has the possibility to cause the phenomenon of so-called anomalous seismic intensity regions. In addition to such a viewpoint, in the 1981 Seismic Design Review Guide and earlier editions, there was a possibility of inviting misunderstanding as to what kinds of earthquakes were postulated because it did not specify, for example, if inside slab earthquakes were considered or not. Hence, in the 2006 edition, with regard to earthquakes generating earthquake ground motions and that can reasonably be regarded as having the possibility of exerting significant impacts on facilities, it was decided that the classification of these earthquakes shall be conducted considering the modes of occurrence.

Earthquakes are generally classified as illustrated in Fig. 7.4. However, since the definition of terms often vary depending on researchers or intended use, the following definition was given in the 2006 edition.

- (1) Inside island earth’s crust earthquake (or inland crustal earthquake): an earthquake that is said to occur in the shallow upper crustal portion, ranging to about 20 km in depth, of a continent side plate. The inside island earth’s crust earthquake refers to such an earthquake occurring in this upper crustal seismogenic layer, and it also includes an earthquake occurring somewhat offshore.
- (2) Inter-plate earthquake: an earthquake occurring at the interfacial plane where two adjacent plates contact.
- (3) Inside oceanic plate earthquake (or oceanic intraplate earthquake): an earthquake occurring in the interior of a subducting or subducted oceanic plate. It is broadly classified into an “earthquake within a subducting oceanic plate” that occurs in the vicinity of the trench axis or somewhat offshore from there, and an “earthquake within a subducted oceanic plate (so-called inside slab (or intraslab) earthquake)” that occurs in an area on the continent side of the vicinity of the trench axis.

After classifying earthquakes as described above, earthquakes whose sources cannot be identified in advance shall also be taken into consideration as earthquakes conceivably exerting significant impacts on the site, as stated in the beginning of this section, in addition to earthquakes identified through the examinations described below.

Previously, earthquakes based on seismotectonics were also listed. The geologic structures in the Japanese Islands and the surrounding areas were classified based on similarity or difference in the modes of earthquake occurrence, and the maximum expected scale of earthquake was determined for each structure. This had played an

important role in selecting marginal earthquakes in the past when sufficiently credible catalogs of past earthquakes with historical records and distribution maps of active faults, etc. were not available [32]. However, such data are sufficiently credible now, and it is possible to examine marginal earthquakes directly using the catalogs of past earthquakes with historical records and active faults. In addition, seismotectonics may differ at the area surrounding the site, and therefore care should be taken in applying it invariably. Nevertheless, since the concept of seismotectonics takes note of the regionality, etc. as to how earthquakes occur, it is important to refer to the latest knowledge on seismotectonics.

[5] Examinations

As stated above, it is very important, as a step for seismic design, to select an earthquake that exerts significant impacts on a site and determine the earthquake ground motion that this earthquake generates at the site as the design basis earthquake ground motion. For this purpose, it is necessary to assume, based on appropriate information, the characteristics of a seismic source fault of the earthquake to be investigated and determine the propagation characteristics of the ground appropriately. Presented below is how examinations on earthquakes in the past with historical records and geologic survey are conducted to determine the characteristics of seismic source faults.

Before going into the main subject, an important perspective that should be understood in conducting examination will be touched on. It should be noted that these examinations are conducted for the purpose of “evaluation,” not for “specification.” In the case of an active fault, for example, if there is what appears to be an active fault, although it is important to judge whether it really is an active fault, it is not necessary to stick to specifying its length, etc. It is important to evaluate rationally on the safe side what is the level of earthquake that this active fault would generate as a single active fault. The examination aims to collect information for this purpose. When considering the risks pertaining to nuclear reactor facilities, it is a matter of course to introduce the latest technology and exercise utmost efforts in the examination and the evaluation based on the examination results. However, as long as man does these, oversight in examination and errors in evaluation will be unavoidable. Hence, in determining an earthquake exerting significant impacts on the site and the earthquake ground motion due to this earthquake, it is presupposed that utmost efforts shall be primarily exercised in determining an “earthquake ground motion whose source to be identified” by conducting detailed examinations. However, in preparation for an earthquake in the vicinity of the site that has still failed to be evaluated, a “site specific earthquake ground motion whose source not to be identified” shall be taken into consideration.

Now, let’s get back to the main subject. How detailed should examinations be conducted? The Western Tottori-ken Earthquake, which occurred on October 6, 2000, provides a significant lesson. The source depth and magnitude of this earthquake were about 10 km and $M_j = 7.3$, respectively. Here, M_j represents the Japan Meteorological Agency’s earthquake magnitude, which is the notation system for magnitude contrived in such a way that the scale of earthquakes occurring in Japan

can be expressed properly. The type of active fault in this earthquake was a lateral fault, which is considered to be relatively difficult in identification. The following items were pointed out as issues in estimating the characteristics of seismic source faults [33].

- (1) Although it was a strong earthquake with a shallow source, the active fault had not been mapped in the existing literature.
- (2) Although the upper end of the seismic source fault was shallow, no obvious surface earthquake fault had been confirmed.

Hence, bibliographic survey, aerial photograph interpretation, surface geologic survey along lineaments, trenching, geophysical prospecting (elastic wave reflection survey along a survey line perpendicular to the fault), which will be described below, etc. were conducted. Consequently, it was found that, even in the case of an ambiguous active fault, it was possible to determine whether it was an active fault having a certain potential for earthquakes if highly accurate examinations were conducted in a proper sequence. In particular, the importance of geophysical prospecting was recognized. Here, the geophysical prospecting is an approach for elucidating underground structures by measuring values pertaining to physical phenomena above or below the earth's surface, such as the geomagnetic field and gravity, or artificially causing physical phenomena from above the ground to below the surface (for example, applying electric currents to the underground, or generating vibrations on the surface by a seismic vibrator to make them propagate underground as elastic waves), measuring their responses, and analyzing the data obtained. Among these, the elastic wave reflection survey is a method for elucidating underground structures, as shown in Fig. 7.6. In this method, artificial vibrations are generated on the earth's surface, and when these vibrations (called elastic waves, since they propagate through the ground, which is an elastic body) travel underground, they are reflected or refracted on interfaces referred to as stratum boundaries, where the physical properties (e.g., P-wave velocity and density) of the ground change. Then, by utilizing the phenomenon that these vibrations partially return to the surface, the reflected waves, etc. are analyzed, thereby providing a method for elucidating underground structures. This method enables us to grasp underground structures continuously and visually. Recently, it is becoming possible to grasp them three dimensionally.

Reference [34] provides guidelines as to what kind of examinations should be conducted and how earthquakes should be postulated. The content of this reference will be described below. Table 7.4 summarizes the content of examination.

(1) Grasping the Modes of Occurrence for the Earthquakes Occurring in the Area Surrounding a Site

Existing research results, such as stress fields and underground structures that can be obtained by geophysical methods, shall be investigated and collected for grasping the modes of earthquake occurrence in the area surrounding a site.

Table 7.4 Content of examination of earthquakes occurring in the area surrounding a site^{*1}

Survey/examination targets	Content of survey/examinations	
Active faults at the site or in the area surrounding the site	Area surrounding the site ^{*2}	Site and its vicinity ^{*3}
	The following surveys/examinations and explorations in the area surrounding the site for grasping the distribution of active faults, etc. (1) Bibliographic survey (2) Tectonic relief geomorphologic feature examination (3) Surface geologic examination (4) Marine sonic examination, etc. (5) Trenching, geophysical prospecting, etc., if needed	Highly accurate surveys/examinations at the site and in its vicinity for minutely grasping the distribution of active faults, etc. including ambiguous or small-scale tectonic relief geomorphologic features by appropriately combining the following surveys/examinations and explorations: (1) Land-based and sea-based survey/examination methods in the area surrounding the site, as noted in the left column (2) Detailed geologic and geophysical examinations through boring survey, trenching, geophysical prospecting, etc.
Past and present situations of earthquake occurrence	(1) Surveys/examinations on the earthquakes that have exerted or will conceivably exert impacts on the site or its surrounding area, on the basis of various examination materials, seismic observation records at the site, etc. (2) Grasping the situations of damage at and surrounding the site, on the basis of a seismic intensity distribution map, etc.	
Inter-plate earthquakes	Surveys/examinations on the earthquakes occurring along the plate boundaries in the areas around the Japanese islands on the basis of the situations of earthquake occurrence in the area around the site, knowledge from various literature, etc.	
Inside oceanic plate earthquakes	Surveys/examinations on the earthquakes occurring within the oceanic plates in the areas around the Japanese islands on the basis of the situations of earthquake occurrence in the area around the site, knowledge from various literature, etc.	

^{*1} In addition to the surveys/examinations described in the table above, it is also necessary to conduct survey/examination for grasping the modes of earthquake occurrence, as well as investigation and collection of knowledge according to the Headquarters for Earthquake Research Promotion, the Central Disaster Prevention Council, etc.

^{*2} The area surrounding the site refers to the area within 30 km or more from the site, as a guide.

^{*3} The site vicinity refers to the area within 5 km or more from the site, as a guide.

(2) Grasping the Active Faults at the Site and in the Area Surrounding the Site

- (a) In order to grasp the distributions of active faults, active folds, active flexures, etc. (referred to as active faults, etc.) in the area surrounding the site (within 30 km or more from the site, as a guide), a geologic map and a tectonic relief geomorphologic distribution map with a scale of about 1/200,000 shall be created on the basis of the results of bibliographic survey, examination of tectonic relief geomorphologic features including lineaments (referred to as tectonic relief

geomorphologic features), surface geologic examination, marine sonic examination, etc., and also taking into consideration the results of trenching, geophysical prospecting, etc. if needed. In addition, active faults that are more than 30 km away from the site and can conceivably exert significant impacts on the site shall be similarly examined.

- (b) At the site and in the vicinity of the site (within 5 km or more from the site, as a guide), highly accurate examinations shall be conducted for minutely grasping the distributions of active faults, etc. including ambiguous or small-scale tectonic relief geomorphologic features by appropriately combining detailed geologic and geophysical examinations through boring survey, trenching, geophysical prospecting, etc., in addition to land-based and sea-based examination methods in the above-mentioned area surrounding the site. Subsequently, a detailed geologic map and a tectonic relief geomorphologic distribution map with a scale of about 1/25,000 shall be created on the basis of these results. Similar examinations shall also be conducted if there are tectonic relief geomorphologic features extending continuously beyond the 5 km radius area. Here, the 5 km radius is used as a guide because it is contemplated that an earthquake ground motion exceeding 450 Gal (unit of acceleration, 1 Gal = 1 cm/s²) is generated at the site by a magnitude 7-class earthquake when the seismic center distance from the site is less than about 5 km [35].
- (c) With regard to the tectonic relief geomorphologic features distributed at, in the vicinity of, and in the area surrounding the site, the correlation with active faults, etc. shall be evaluated on the basis of the results from geologic survey, and when the correlation with active faults, etc. is negated, the negation shall be based on cogent geographical and geologic reasons.
- (d) Based on the geologic map and tectonic relief geomorphologic distribution map, as well as the results of geologic survey at, in the vicinity of, and in the area surrounding the site, the active faults requiring considerations in order to select investigation earthquakes shall be evaluated. Here, investigation earthquakes refer to earthquakes anticipated to exert significant impacts on the site.
- (e) The active faults, etc. requiring considerations in selecting investigation earthquakes shall be those whose activeness since the late Pleistocene epoch cannot be negated. However, considering that the distributions of late Pleistocene strata are in many cases limited, the evaluation is possible by focusing on whether or not displacement or deformation due to faults can be observed in the strata or on the geographical surfaces, etc. of the last interglacial period.
- (f) The formation ages of strata and geographical surfaces used for evaluating the activities of active faults, etc. shall be evaluated by volcanic ash layers and such methods as comparing terrace surfaces.
- (g) In evaluating active faults, etc., care shall be taken regarding the consistency between land and sea areas.
- (h) With regard to the latest active period, length, displacement per earthquake, etc. of active faults, the evaluation shall be conducted based on the displacement, deformation, etc. of strata or geographical surfaces since the late Pleistocene epoch obtained from the results of various examinations and exploration, such

as tectonic relief geomorphologic feature examination, surface geologic examination, boring survey, trenching, and geophysical prospecting. Furthermore, when a source is postulated, the latest knowledge of geomorphology, geology, and geophysics regarding the fracture (or rupture) process of faults, modes of earthquake occurrence, underground structures, etc. shall be taken into account on the basis of the results of above-mentioned evaluations.

(3) Grasping the Past and Present Situations of Earthquake Occurrence

- (a) Based on various examination materials, observation records of earthquakes at the site, etc., the earthquakes that have exerted, or have conceivably exerted, impacts on the site and surrounding area from past to present shall be examined.
- (b) Based on the seismic intensity distribution map etc. in the examination materials, the situations of damages on the site and the surrounding area shall be grasped.

(4) Grasping the Earthquakes Occurring Between Plates and Within Oceanic Plate

Based on the situations of earthquake occurrences in the area surrounding the site and the knowledge from literature, etc., earthquakes that occur along the plate boundaries and within the oceanic plate in the areas around the Japanese islands shall be investigated.

(5) Reflecting Other Knowledge

The knowledge on earthquakes and earthquake ground motions accumulated by the Headquarters for Earthquake Research Promotion and the Central Disaster Prevention Council shall be investigated and collected.

Based on the information acquired from the results of the above-mentioned examinations, the earthquakes shall be classified into “inside island earth’s crust earthquake,” “inter-plate earthquakes,” “inside oceanic plate earthquakes,” etc., focusing on the modes of earthquake occurrence, etc. For each category, the characteristics of the fault that is postulated as a source shall be studied by reference to the past earthquake activities, microearthquake activities and geophysical knowledge based on the above-mentioned information, and the scale and locations of the earthquakes shall be estimated.

(1) Inside island earth’s crust earthquake

- a. Based on the examination results, a source shall be assumed at an active fault, etc. in the area surrounding the site.
- b. The scale of the earthquake shall be postulated from the length, displacement per earthquake, etc. of the fault. When assumption is made from the length or displacement per earthquake of the fault, a proven empirical formula may also be used. In selecting and applying an empirical formula, while it is necessary to be sufficiently careful about the characteristics, etc. of the empirical formula, the latest knowledge shall be referred to.

(2) Inter-plate earthquakes

- a. Based on the examination results, an inside oceanic plate earthquake that would exert significant impacts on the site shall be postulated.
- b. The scale and location of the earthquake shall be either those of the largest scale and the location of inside oceanic plate earthquakes that has occurred in the area surrounding the site in the past, or shall be decided by referring to the latest knowledge on the scale and locations.

(3) Inside oceanic plate earthquakes

- a. Based on the examination results, an inter-plate earthquake that would exert significant impacts on the site shall be postulated.
- b. The scale and location of the earthquake shall be either those of the largest scale and the location of inter-plate earthquakes that occurred in the area surrounding the site in the past, or shall be decided by referring to the latest knowledge on the scale and locations.
- c. The earthquake shall be configured reflecting the regional aspects of the mechanism, scale, and locations of earthquakes.

(4) Other earthquakes

Earthquakes other than those in the aforementioned categories shall also be considered, if necessary.

Regarding these examined earthquakes, mutual relationship shall be evaluated using empirical methods, etc. by setting basic source parameters (scale, location, etc.), and multiple earthquakes anticipated to exert particularly large impacts shall be selected as “investigation earthquakes.”

Meanwhile, regarding the above-mentioned examinations and accreditation of active faults considered in seismic design, examination of guidelines for safety review is currently underway in the “Committee on the Guidelines for Safety Review Concerning Geology and Ground of Nuclear Power Reactor facilities” of the Nuclear Safety Commission of Japan. As of August 2008, the draft is under review.

[6] Methods for Computing Design Basis Earthquake Ground Motions

Twenty years have passed since the 1981 Seismic Design Review Guide was instituted, and observation networks for large-scale earthquakes have been implemented. Meanwhile, the occurrence frequency of intense earthquakes in the Japanese Islands has been on the rise in recent years. Consequently, a plenty of observation data have been accumulated regarding earthquakes generating strong ground motions. In particular, the observation data in the near field hypocenters (or sources) have come to be obtained.

Clarification of average characteristics pertaining to earthquake ground motions has progressed on the basis of these observation data on earthquakes, and the efforts for improving the empirical equations used for evaluating earthquake ground motions have been pursued. In parallel with the progress in such empirical methods, theoretical methods, i.e., so-called fault model approaches, which evaluate earthquake

ground motions by properly modeling seismic source faults, have also progressed in the research of fracture processes in seismic source faults, and it is becoming possible to reproduce the data of strong ground motions by properly selecting the parameters for describing faults. Recently, also in so-called empirical methods, which evaluate earthquake ground motions by using an empirical formula for attenuation with distance, fault models have come to be utilized to modify the formula. Among the formulae for attenuation with distance based on the fault model is the Kobayashi-Midorikawa equation [36] proposed in 1979.

Against this backdrop, developing an advanced method for determining design basis earthquake ground motions was examined in the course of establishing the 2006 Seismic Design Review Guide. In the previous methods for determining design basis earthquake ground motions for nuclear reactor facilities, the Ohsaki's method noted above in reference to the frequency characteristics of earthquake ground motions, for example, had been mainly used as an empirical formula. Even so, however, if earthquakes to be considered are occurring in the immediate vicinity of the site, it is desirable to examine the geometric expanse and fracture process of faults. Therefore, earthquake ground motions were evaluated, as needed, using a method with a fault model.

As regards the direction of advancement, which would depend on whether or not the evaluation of faults is possible, the “site specific earthquake ground motion whose source to be identified,” and the “earthquake ground motion whose source not to be identified” will be described separately. As a summary, a rough outline of the work flow for determining design basis earthquake ground motions is illustrated in Fig. 7.7. Note that in some cases, the “site specific earthquake ground motion(s) whose source(s) to be identified,” and the “earthquake ground motion(s) whose source(s) not to be identified” are called the “earthquake ground motion(s) formulated with hypocenter(s) (or seismic source(s)) specified for each site” and the “earthquake ground motion(s) formulated without hypocenter(s) (or seismic source(s)) specified, as English translation of each Japanese clause, respectively.

(1) Site Specific Earthquake Ground Motion Whose Source to Be Identified

The evaluation of earthquake ground motions using a fault model, which is a theoretical approach, is highly rational because the logic can be clearly forwarded, and also considering the accountability and transparency, it may be regarded as the desirable first candidate. However, in those days when advancement to the 2006 edition was under study, it was pointed out that the evaluation technique using a fault model was still in the development stage in the following points.

- (a) In the evaluation of earthquake ground motions on the short period side, there were matters that had not been thoroughly clarified yet [38].
- (b) In the spectra of design earthquake ground motions, there was possibility that valleys would emerge in certain frequency bands. It might be necessary to contrive a means to unfailingly cover the entire frequency range [39]. In the response spectrum method, which was one of the conventional empirical methods, design basis earthquake ground motions had been determined in such

a way that earthquake ground motions used were enveloped. Since several kinds of naturally occurring seismic waves were used in those days, only three or four of these waves tended to produce valleys in certain frequency bands. In order to sufficiently ensure the safety of components whose natural periods fall within the above frequency bands, spectra have come to be determined in such a way that valleys are not formed as a result of envelop delineation [40]. It should be noted that evaluation using a fault model may potentially cause this problem.

With reference to (b), although the response spectrum method handles averaged phenomena and has difficulty in predicting specific seismic waves, it is robust in the sense that it covers the entire frequency range unfailingly. In this way, in view of the technological level and past experiences pertaining to the approach utilizing the response spectrum method, it has been decided that, both of the following shall be carried out as methods for computing design basis earthquake ground motions for each of the selected design earthquakes, and the earthquake ground motion generated by each of these earthquakes shall be determined for vertical as well as horizontal directions.

- (i) Evaluation of earthquake ground motions based on response spectra, which is an empirical method using a formula for attenuation with distance.
- (ii) Evaluation of earthquake ground motions by the method using a fault model.

In the evaluation of earthquake ground motions based on response spectra, the response spectra shall be evaluated for each of the earthquakes anticipated to exert significant impacts on the site (referred to as “investigation earthquakes”) by using appropriate approaches, and the response spectra for design shall be configured on the free surface of the base stratum at the site on the basis of these results, appropriately reflecting thereto the characteristics of earthquake ground motions, such as the duration time of earthquake ground motion, temporal changes in the amplitude-enveloping curve, etc. Here, the free surface of the base stratum refers to a hypothetical free surface where there is no surface layer or structures on the base stratum plane, which is assumed as a base stratum surface without substantial difference in height, nearly flat, and having considerable expanse. Generally, the “base stratum” as used here shall be a hard ground where the velocity of shear wave is larger than $V_s = 700$ m/s and which has not undergone significant weathering. The shear wave is identical to a transverse wave. Meanwhile, the reason for ripping off surface layers, etc. in the above assumption is that, if surface layers, etc. are present as upper layers on the free surface of the base stratum, the seismic waves to be determined on the free surface of the base stratum will be altered due to interactions of these layers with the free base stratum. The aforementioned assumption shall be made to avoid such a situation.

In the evaluation of earthquake ground motions based on the fault model approach, the parameters associated with the characteristics of a source shall be set for each investigation earthquake using appropriate methods. The earthquake ground motions on the free surface of the base stratum at the site shall be evaluated using these parameters.

Meanwhile, as mentioned above, although the Ohsaki’s spectrum method had been used conventionally as a response spectrum method, the Seismic Design Special Committee’s spectrum method, which has evolved substantially from the former, has been proposed [41]. The Seismic Design Special Committee’s spectrum is referred to as such, since it has been proposed and discussed in the Special Committee on Seismic Design for Nuclear Power Generation of Japan Electric Association. This spectrum has been improved compared with the “Ohsaki’s spectrum” as follows.

- Since the database of earthquake ground motions in the hard base stratum, which was scarce in data in the 1980s, have become profuse in data, the Seismic Design Special Committee’s spectrum is based on this database.
- Empirical equations with a clear physical meaning have been adopted. Specifically, effects due to the planar spread of a fault plane are taken into consideration by introducing the concept of the equivalent seismic center distance.
- Response spectra in the seismic base stratum are configured and corrected for the effect of ground amplification in determining the response spectra on the free surface of the base stratum. Vertical motions are also evaluated. That is, horizontal and vertical (perpendicular) earthquake ground motions on the free surface of the base stratum are determined as follows. Specifically, the horizontal earthquake ground motion in the seismic base stratum is used in common, and it is multiplied by the ground amplification factors for horizontal and vertical motions. Following Ohsaki’s method, the response spectra in the seismic base stratum provide for the maximum response velocities at specific points (these are called control points) on a response spectral diagram according to the seismic magnitude and the distance from the source. The response spectra are applied directly to inter-plate earthquakes, and with corrections for inside island Earth’s crust earthquakes and somewhat deeper inside slab earthquakes.

(2) Earthquake Ground Motion Whose Source not to Be Identified

In cases where sources cannot be specified, it is conceivable that inside island earth’s crust earthquakes up to magnitude 7.3 may potentially occur. Therefore, response spectra (in both horizontal and vertical (perpendicular) directions) on the free surface of the base stratum shall be configured in accordance with the records of observation in the vicinity of the sources for the past inside island earth’s crust earthquakes not exceeding magnitude 7.3 in Japan and overseas, and the design basis earthquake ground motion shall be determined from these spectra by properly taking various characteristics of earthquake ground motions into consideration.

[7] Evaluation of the Probability of Exceedance Occurrence in Response Spectra

In both the response spectrum method, which is an empirical approach as described above, and the fault model, which is a theoretical approach, earthquake ground motions are developed deterministically based on average characteristics. However, there exists uncertainty in modeling, etc. The formula for attenuation with distance, for example, is obtained from various combinations of earthquakes and observation points. Variations in data would be caused by the characteristics of seismic sources,

the characteristics of propagation paths from the source to the respective observation points, and the site characteristics at the respective observation points. Although the attenuation formula itself is generally obtained as a regression formula, it is becoming important to estimate the standard deviation, etc. Such variations are largely classified into two types: one is the intrinsic variation as natural phenomena (corresponding to randomness), while the other is the uncertainty in modeling in which values cannot be determined uniquely due to the lack of information.

Therefore, even if a certain marginal earthquake ground motion is prescribed deterministically, we cannot deny the possibility that earthquake ground motions exceeding this may occur at the site. It is desired that certain estimation could be conducted with regard to the probability of occurrence. This corresponds to the assessment of “residual risks.” Hence, with regard to the “site specific earthquake ground motion whose source to be identified” and the “earthquake ground motion whose source not to be identified,” it is desirable to grasp how high the exceedance probability corresponding to the response spectrum of each determined earthquake ground motion would be.

Generally, at a certain geographical point, the relationship between the intensity of the earthquake ground motion and the probability of exceedance within a certain period of time is expressed as a seismic hazard curve and the computation of this curve is referred to as seismic hazard assessment. Roughly speaking, the assessment shall be conducted as follows [42].

- (1) At an evaluation point, the seismic activity in the surrounding area shall be evaluated. Subsequently, potential seismic source faults shall be identified.
- (2) A fault model shall be configured for each fault.
- (3) The probability (or frequency) of earthquake occurrence, earthquake scale, etc. shall be estimated. The Gutenberg-Richter (G-R) formula indicated below is frequently used for the frequency of earthquake occurrence, although it is a classical formula.

$$\log_{10} N(M) = a - bM \quad (7.4)$$

Here, M is magnitude, and $N(M)$ is the annual average frequency of occurrence (or the number of occurrences) of earthquakes with magnitude larger than or equal to M .

- (4) A probabilistic model shall be configured for estimating the intensity of earthquake ground motion from the scale and distance of an earthquake. The evaluation is often conducted with a formula for attenuation with distance and its variability.
- (5) For each earthquake, the probability that the intensity of an earthquake ground motion exceeding a certain value within a predetermined period of time as a result of the earthquake shall be evaluated; this process is repeated as many times as the number of earthquakes to be evaluated. Finally, the probabilities that the intensities of earthquake ground motions exceeding certain values at

least once within the predetermined time period shall be derived, taking all the earthquakes into consideration.

7.3.4 *Principle of Seismic Design*

As an important point in considering “How safe is safe enough?”, the classification of importance in the seismic design, i.e., classification in accordance with the importance of safety functions that are required for facilities during earthquakes, was described first of all in Sect. 7.3.2. Then, explanation was given in Sect. 7.3.3 as to what kind of earthquake ground motions should be postulated for the seismic design of nuclear reactor facilities, and what kind of examinations should be conducted for that purpose. Rational consideration on “How safe is safe enough?” signifies that the seismic design of facilities with pivotal safety functions shall be conducted with regard to the strongest earthquake ground motion that may potentially occur, because the ensuing consequences would be serious if by any chance nuclear reactor facilities were failed by such earthquakes. Meanwhile, the seismic design of facilities that, even if they are failed as a result of an earthquake, only exert impacts on the general public or workers comparable to those due to general industry facilities, shall be conducted with regard to earthquake ground motions having intensity comparable to those used for the design of general industry facilities. The seismic design principle stipulates what shall be the strength of earthquake ground motions to be postulated in accordance with the importance of seismic safety functions that are required for facilities, and in what conditions the facilities shall ensure their safety functions against seismic forces due to these earthquake ground motions.

Although the basic concept of seismic design principle in the 2006 Seismic Design Review Guide has not changed significantly from the previous edition, it is stipulated as described below while reflecting the latest knowledge as described in previous sections and improvements to and progress in seismic design technology.

(1) **Basic Principle**

Facilities shall satisfy the basic policy on seismic design for respective seismic design classes, as indicated below.

- (1) Each facility in Seismic class S shall be able to maintain its safety functions against the seismic force due to the design basis earthquake ground motion S_s . In addition, it shall withstand the seismic force due to the elastically dynamic design earthquake ground motion S_d indicated below or the static seismic force indicated below, whichever is greater.
- (2) Each facility in Seismic class B shall withstand the static seismic force indicated below. In addition, regarding the facility with a risk of resonance, its consequence shall be examined.
- (3) Each facility in Seismic class C shall withstand the static seismic force indicated below.

- (4) In each of the aforementioned items, a facility that belongs to a higher class shall not be failed by the spill-over effect of failure of facilities belonging to lower classes.

(2) Methods for Computing Seismic Forces

Computation of seismic forces for use in the seismic design of facilities shall be based on the methods indicated below.

- (1) The seismic force due to the design basis earthquake ground motion S_s

The seismic force due to the design basis earthquake ground motion S_s shall be computed for horizontal and vertical directions in an appropriate combination by applying the design basis earthquake ground motion S_s .

- (2) The seismic force due to the elastically dynamic design earthquake ground motion S_d

The elastically dynamic design earthquake ground motion S_d shall be configured on the basis of the design basis earthquake ground motion S_s in accordance with engineering judgment. In addition, the seismic force due to the elastically dynamic design earthquake ground motion S_d shall be computed for horizontal and vertical directions in an appropriate combination.

- (3) The static seismic force

Computation of the static seismic force shall be based on the methods indicated below.

- (i) Buildings and structures

Horizontal seismic force shall be computed by multiplying the seismic story shear force coefficient (or earthquake layer shearing force coefficient) C_i by the coefficient corresponding to the importance classification of the facility as shown below, and further multiplying by the weight of the relevant story and above.

Seismic class S 3.0

Seismic class B 1.5

Seismic class C 1.0.

Here, the seismic story shear force coefficient C_i shall be the value derived by assuming 0.2 for the standard shear coefficient C_0 , and considering the vibration characteristics of the building or structure, the ground type, etc. The detail of seismic story shear force coefficient is introduced in Item [1] in Sect. 7.5.1.

For facilities in Seismic class S, horizontal and vertical seismic forces shall be assumed to act simultaneously with a combination of adverse directions. Vertical seismic force shall be computed assuming the seismic intensity of 0.3 as a standard and considering the vibration characteristics of the building or structure, the ground type, etc. However, the vertical seismic intensity shall be assumed to be constant in the height direction.

Further, it is stipulated that the horizontal bearing capacities (in other words, retained horizontal strength, or horizontal load-carrying capacity) of relevant buildings and structures shall be confirmed to possess adequate safety margins against the required horizontal bearing capacity in accordance with the importance of the facilities. The concept of required (or requisite) horizontal bearing capacity is introduced in detail in Item [1] in Sect. 7.5.1.

(ii) Components and piping systems

The seismic force for each seismic class shall be derived from the seismic intensity obtained in the following way: the horizontal seismic intensity is first computed by multiplying the seismic story shear force coefficient C_i noted above by the coefficient corresponding to the importance of the facility, and then this horizontal seismic intensity and the vertical seismic intensity described in (i) above are both increased by 20%.

Furthermore, horizontal and vertical seismic forces shall be assumed to act simultaneously with a combination of adverse directions. However, the vertical seismic intensity shall be assumed to be constant in the height direction.

In the description above, the term “withstand seismic force” means that the design is conducted generally within the elastic range of each facility as a whole against the assumed seismic force. In this case, the design within the elastic range means that the stress analysis is performed assuming each facility as an elastic body, and the stress in each portion of the facility is kept below the allowable limit described below. More specifically, although this allowable limit generally means an elastic limit, it is not necessarily the elastic limit in its strict sense, and it would be sufficient if the stress can be generally kept within the elastic limit for each facility as a whole, while allowing cases where the elastic limit may be exceeded locally.

The seismic force due to the design basis earthquake ground motion S_s and that due to the elastically dynamic design earthquake ground motion S_d shall be computed by dynamic analysis in both horizontal and vertical (i.e., perpendicular) directions, because the design basis earthquake ground motion S_s is determined as earthquake ground motions in horizontal and vertical directions on the free surface of the base stratum at the site.

In addition, although they will not be included as a target of safety regulations for some time to come, “residual risks” shall be assessed considering the possibility of earthquake ground motions exceeding S_s , partly because it is stipulated that, in determining the design basis earthquake ground motion S_s , the exceedance probability for that motion shall be referenced.

The discussions above are summarized in Table 7.5.

7.3.5 Load Combinations and Allowable Limits

In conducting the seismic design in accordance with the seismic design principle mentioned in the previous section, if we suppose an earthquake occurring with no

Table 7.5 Principle of seismic design (seismic forces and basic principles) and outline of load combinations and allowable limits

Seismic importance classification		Seismic force to be assumed	Basic principle of allowable state	Load combination	Allowable limit
Buildings and structures	S	Dynamic seismic force due to the design basis earthquake ground motion S_d^{*1}	To keep safety functions required	Both of the following loads should be combined: (i) Stational loads (SL) and operational loads (OL) (ii) Seismic force due to S_e	(a) To have a sufficient margin for the deformation capability as a whole structure (b) To have an adequate margin for the ultimate load carrying capacity
		The greater of dynamic seismic force due to the elastically dynamic design earthquake ground motion S_d^{*2} or static seismic force $3.0C_d/W^{*3}$, conceptually expressed as $\max(S_d, 3.0C_d/W)$	To withstand (namely, to keep within the elastic regime as a whole)	Both of the following loads should be combined: (i) SL and OL (ii) Dynamic seismic force due to S_d or static seismic force specified in the left hand column	Allowable stress degree (or unit stress) provided in codes and standards that are recognized as appropriate for ensuring safety (for example, short-term allowable stress degrees given in the Building Standards Act)
	B	Static seismic force $1.5C_d/W^{*4}$	To withstand, and if a facility has the possibility of resonating with an earthquake, to investigate its possibility and effects.	Both of the following loads should be combined: (i) SL and OL (ii) Static seismic force specified in the left hand column	Allowable stress degree (or unit stress) provided in codes and standards for stresses produced by the application of the combined loads specified in the left hand column
	C	Static seismic force $1.0C_d/W^{*4}$	To withstand	Both of the following loads should be combined: (i) SL and OL (ii) Static seismic force specified in the left hand column	Allowable stress degree (or unit stress) provided in codes and standards for stresses produced by the application of the combined loads specified in the left hand column
	S	Dynamic seismic force due to the design basis earthquake ground motion S_d^{*1}	To keep safety functions required	Both of the following loads should be combined ^{*6} : (i) Loads at normal operation, anticipated operational occurrence and accident (ii) Dynamic seismic force due to S_e	It is required not to damage functions of the facility as a result of causing an excessive deformation, cracking, failure, etc., even if major parts of structures are yielded, and plastic deformations are developed against stresses produced by application of the combined loads. ^{*7}
		The greater of dynamic seismic force due to the elastically dynamic design earthquake ground motion S_d^{*2} or static seismic force (the horizontal seismic intensity is assumed to be obtained by multiplying the earthquake layer shearing force coefficient C_d by 3.0 (3.0 is the factor depending on the seismic importance. The static seismic force is determined from the seismic intensity obtained from both of the horizontal seismic intensity as mentioned above, and the vertical seismic intensity C_v as computed according to the note below ^{*3} , by adding 20% of them, respectively) ^{*5} , conceptually expressed as $\max(S_d, 3.6C_d/W)$	To withstand	Both of the following loads should be combined ^{*6} : (i) Loads that are developed at normal operation and anticipated operational occurrence (ii) Dynamic seismic force due to S_d or static seismic force given in the left hand column	Yield strength or a stress to assure the equivalent safety, for stresses produced by application of the combined loads

(continued)

Table 7.5 (continued)

Seismic importance classification		Seismic force to be assumed	Basic principle of allowable state	Load combination	Allowable limit
Components and piping systems	B	Static seismic force $1.8C_iW$ (the horizontal seismic intensity is assumed to be obtained by multiplying the earthquake layer shearing force coefficient C_i by 1.5. The static seismic force is determined from the seismic intensity obtained from both of the horizontal seismic intensity as mentioned above, and the vertical seismic intensity C_v as computed according to the note below ^{*3} , by adding 20% of them, respectively) ^{*5} .	To withstand, and if a facility has the possibility of resonating with an earthquake, to investigate its possibility and effects	Both of the following loads should be combined ^{*6} : (i) Loads at normal operation and anticipated operational occurrence (ii) Static seismic force given in the left hand column	Yield strength or a stress to assure the equivalent safety, for stresses produced by application of the combined loads
	C	Static seismic force $1.2C_iW$ (the horizontal seismic intensity is assumed to be obtained by multiplying the earthquake layer shearing force coefficient C_i by 1.0. The static seismic force is determined from the seismic intensity obtained from both of the horizontal seismic intensity as mentioned above, and the vertical seismic intensity C_v as computed according to the note below ^{*3} , by adding 20% of them, respectively) ^{*5} .	To withstand	Both of the following loads should be combined ^{*6} : (i) Loads at normal operation and anticipated operational occurrence (ii) Static seismic force given in the left hand column	Yield strength or a stress to assure the equivalent safety, for stresses produced by application of the combined loads

[Note]

*1: The seismic force is computed as a proper combination of the horizontal direction and the vertical direction (in the new NRA regulation, the directions are clearly described as “the two horizontal directions and the vertical direction”).

*2: The elastically dynamic design earthquake ground motion (or the elastic design earthquake ground motion) is established by multiplying the design basis ground motion by a specific value as the response spectrum ratio β , namely $S_d = \beta S_n$, $\beta < 1$. The seismic force due to S_d should be calculated as a proper combination of the horizontal direction and the vertical direction.

*3: 1) The horizontal static seismic force is calculated by multiplying the weight at and above the layer concerned by the earthquake layer shearing force coefficient C_i and the factor in accordance with the seismic importance of the facility (3.0 for Seismic class S facility). The horizontal and vertical seismic forces acting on a Seismic class S facility should be combined simultaneously in their adverse directions.

2) The vertical seismic force is calculated using the vertical seismic intensity C_v that is derived from the standard seismic intensity of 0.3 (in the new NRA regulation, 0.3 or more) and by taking into consideration vibration characteristics, etc. of the building and structure. The vertical seismic intensity should be constant in the height direction.

3) Further, the buildings and structures should be designed to have the horizontal load-carrying capacity exceeding the requisite horizontal load-carrying capacity with an adequate safety margin.

*4: 1) The horizontal static seismic force is calculated by multiplying the weight at and above the layer concerned by the earthquake layer shearing force coefficient C_i and in addition the factor “ n ” depending on the seismic safety importance of the facility ($n=1.5$ for Seismic class B facility, and $n=1.0$ for Seismic class C).

2) Further, the buildings and structures should be designed to have the horizontal load-carrying capacity exceeding the requisite horizontal load-carrying capacity with an adequate safety margin

*5: The horizontal and vertical seismic forces acting on a Seismic class S facility should be combined simultaneously in their adverse directions. The vertical seismic intensity should be constant in the height direction.

*6: If the probability of simultaneous occurrence of the earthquake and an accident event is judged to be extremely low, taking into account the occurrence frequency of the accident event, its duration period and the exceedance probability of the earthquake, it need not combine the seismic force due to the earthquake with loads produced by the accident event.

*7: Allowable limits for dynamic components (or active components), etc. should be function-maintaining acceleration, etc. that should be confirmed by verification tests and so on for the response by S_n , because the functions required should be kept for the response.

choice of time and place, then what would be the operating condition of a nuclear power station and what would be the intensity of the earthquake occurring there that should be postulated? In addition, what would be the specific meaning of “withstanding” and “maintaining safety functions” that the seismic design principle requires? The answers to these questions have been provided as “load combinations and allowable limits” in the 2006 edition, as well as the 1981 Seismic Design Review Guide. Although the basic concept does not differ from that of the 1981 edition described in Item [3] in Sect. 7.2.3, the following points, implicitly postulated in the 1981 edition, have been clearly stated in the 2006 edition.

- (1) Probabilistic consideration on the combination of strong earthquakes with “anticipated operational occurrences” and “accidents”

Although it is a matter of course to assume the occurrence of strong earthquakes during normal operation, in view of the fact that “anticipated operational occurrences” and “accidents” are limited in terms of the duration of the events, it is important to consider in terms of the probability of combination whether the occurrence of a strong earthquake needs to be assumed during the occurrence of these events. Consideration in terms of the probability of combination had been tacitly practiced even in the previous Regulatory Guide for Reviewing Seismic Design, as mentioned in Item [1] (1) in Sect. 7.2.3. The 2006 Seismic Design Review Guide stipulates that the probability of exceedance occurrence regarding the design basis earthquake ground motion S_s shall be evaluated, as noted in Item [7] in Sect. 7.3.3. Hence, with regard to the “accident” events, the necessity for the combination shall be assessed in consideration of the occurrence probability and duration of these events, as well as from the probability of exceedance occurrence regarding the design basis earthquake ground motion S_s .

- (2) For dynamic (or active) components, etc., the concept for the maintenance of functions when impacted by the design basis earthquake ground motion S_s

In the 2006 Seismic Design Review Guide,” the “load combinations and allowable limits” are stipulated as follows on the basis of the above description, the unification of the design basis earthquake ground motions, the rationalization of the importance in the seismic design, etc. In the description below, commentaries are added as well.

“7. Load combinations and allowable limits

In evaluating the validity of the design principle for seismic safety, the basic concept of load combinations and allowable limits that should be considered is indicated below.

- (1) Buildings and structures
 - (a) Buildings and structures in Seismic class S
 - (i) Combinations with the design basis earthquake ground motion S_s , and allowable limits

For the combination of loads, consisting of normal loads (i.e., those that always act) and operating loads (i.e., those that act during operation), and the seismic force due to the design basis earthquake ground motion S_s , the relevant building or structure shall have a sufficient margin of deformation capability (deformation at the ultimate strength) as a whole structure, and it shall possess an adequate safety margin compared to the ultimate strength of the building or structure.

- (ii) Combinations with the elastically dynamic design (or elastic design) earthquake ground motion S_d , etc., and allowable limits

Normal loads and operating loads are combined with the seismic force due to the elastic design earthquake ground motion S_d or the static seismic force, and the allowable limit for the stresses resulting from the combination shall be the allowable unit stress (or stress degree) specified in the codes and standards that are recognized as appropriate from the standpoint of safety.

- (b) Buildings and structures in Seismic classes B and C

Normal loads and operating loads are combined with the static seismic force, and the allowable limit for the stress resulting from the combination shall be the allowable unit stress described in (1) (ii) above.

- (2) Components and piping systems

- (a) Components and piping systems in Seismic class S

- (i) Combinations with the design basis earthquake ground motion S_s , and allowable limits

Respective loads generated during normal operation, anticipated operational occurrences and accidents are combined with the seismic force due to the design basis earthquake ground motion S_s , and the functions of a facility shall not be affected by the stresses resulting from the combinations through the occurrence of excessive deformation, cracks, failure, etc., even if considerable portions of the structure undergo yielding and plastic deformation. Meanwhile, with regard to active components, the allowable limits shall be the acceleration for retaining of function or the like, which have been confirmed for the responses caused by the design basis earthquake ground motion S_s in verification tests, etc.

- (ii) Combinations with the elastic design earthquake ground motion S_d , etc. and allowable limits

Respective loads generated during normal operation, anticipated operational occurrences, and accidents are combined with the seismic force due to the design basis earthquake ground motion S_d or the static seismic force; the allowable limits for the resulting stress shall correspond to the yield stress or the stress that ensures the equivalent level of safety.

(b) Components and piping systems in Seismic classes B and C

Loads during normal operation and anticipated operational occurrences are combined with the static seismic force, and the allowable limit for the resulting stress shall correspond to the yield stress or the stress that ensures the equivalent level of safety. (Commentary).

- With regard to the “respective loads generated during anticipated operational occurrences and accidents,” the loads emerging from an event that may potentially be caused by an earthquake and the loads due to an event that lasts long once it occurs although its occurrence due to an earthquake may not be probable shall be considered in combination with seismic forces.

However, even for the case of the “loads generated during accidents,” if the possibility of simultaneous occurrence is extremely small in view of the relationship between the occurrence probability/duration of the accident event and the exceedance probability of the earthquake ground motion, it is not necessary to consider the loads generated by such events in combination with seismic forces.

- The prescription that the allowable limits for buildings and structures against the combinations with the elastically dynamic design earthquake ground motion S_d shall be the “allowable unit stress specified in the codes and standards that are recognized as appropriate from the standpoint of safety” corresponds specifically to that in the Building Standards Act, etc.
- The “ultimate strength” mentioned in the item regarding the combinations with the design basis earthquake ground motion S_s for buildings and structures has the following meaning. When the load on a structure is increased gradually, the load would lead to an ultimate state where deformation or strain has increased significantly. The ultimate strength means the bounding maximum bearing load leading to this ultimate state.
- With regard to the allowable limits for components and piping systems, the basic concept was that the allowable limits “for the resulting stress shall correspond to the yield stress or the stress that ensures the equivalent level of safety,” and specifically, the “Technical Standards for Nuclear Power Generation Equipment” provided in the Electricity Business Act, etc. correspond to this.”

7.4 Strengthening of Seismic and Tsunami-Resistant Design Reflecting the 2011 Tohoku Oki Earthquake

In Japan, occurrence frequencies of large-scale (or strong) earthquakes have been increasing since the Southern Hyogo Prefecture Earthquake, which occurred on January 17, 1995. Under such circumstances, the 2007 Niigata Chuetsu Oki (or Chuetsu-oki) (Niigata Chuetsu Oki: literally, off the coast of Chuetsu area in Niigata Prefecture) Earthquake struck Niigata and the surrounding areas on July 16, 2007. Its moment magnitude was M_w 6.6, and the corresponding Japan Meteorological Agency Magnitude was M_j 6.8. The acceleration of this earthquake was recorded

more than twice the design basis earthquake ground motion that was specified before the earthquake, on the base mats of the reactor buildings at the TEPCO Kashiwazaki-Kariwa nuclear power station (hereinafter, refer to as “TEPCO K-K nuclear station”). Currently, the full name of TEPCO is changed to the Tokyo Electric Power Company Holdings, Inc. The acceleration more than twice the design basis earthquake ground motion was caused due to special seismic source characteristics and underground structure.

Afterward, the 2011 Tohoku Oki Earthquake occurred on March 11, 2011, and huge tsunamis triggered by this large-scale earthquake hit the Pacific coast of eastern Japan and other areas. This earthquake occurred along the plate boundary where the oceanic Pacific Plate subducts beneath the continent side Eurasian Continent. Near the Japan Trench, the continent side plate jerked upward abnormally larger than expected, causing very large tsunamis attacking the Pacific coast of eastern Japan and other areas. These earthquake and tsunamis caused the severe accident at the Fukushima Daiichi nuclear power station, resulting in release of a large amount of radioactive materials, which are consisting of fission products, to the environment. Many local residents had to evacuate and even in 2018, the restoration from the disaster and their returns have not yet completed.

Responding to the Fukushima Daiichi nuclear accident, safety regulations including regulations on tsunami-resistant design as well as seismic design were drastically modified in Japan. In particular, the stages 4 and 5 safety measures as the defense-in-depth safety design approach became newly subject to regulation, as briefly mentioned in Chap. 1. Before the accident, in the safety regulations in Japan, only the stages 1 to 3 were regulated, and countermeasures against severe accidents as the stage 4 safety measures were entrusted to voluntary activities of licensees.

The Reactor Regulation Act (the Act for Regulation on Nuclear Source Material, Nuclear Fuel Material and Nuclear Reactors) stipulates rules for the stages 1 to 4 safety measures. As nuclear reactor facilities, facilities for design basis events have safety functions for taking the stages 1 to 3 safety measures, and are functioned at design basis events including design basis accidents, while facilities for coping with severe accidents, etc. have functions for taking the stage 4 safety measures for coping with severe accidents, etc. Regarding safety requirements on seismic design of facilities for design basis events, the rules remain basically unchanged from those specified in the 2006 Seismic Design Review Guide. Several changes have been made in response to the 2007 Niigata Chuetsu Oki Earthquake and other strong earthquakes. However, safety requirements on tsunami-resistant design drastically changed. Before the Fukushima Daiichi nuclear accident, the tsunami was treated as an earthquake-accompanying phenomenon, and the related rules are briefly stipulated in the 2006 Seismic Design Review Guide. Requirements on seismic and tsunami-resistant design are newly regulated after the accident.

In the following, four items as key issues on how to strengthen seismic and tsunami-resistant design are discussed. The items are as follows:

- (1) How to cope with strong earthquakes that we experienced between the time of establishment of the 2006 Seismic Design Review Guide and the 2011 Tohoku Oki Earthquake,
- (2) Identification of detailed mechanisms, etc. of the 2011 Tohoku Oki Earthquake and following huge tsunami,
- (3) How to strengthen and change the current seismic and tsunami-resistant design methodologies in order to reflect lessons learned from the Fukushima Daiichi nuclear accident caused by the earthquake and tsunami, and
- (4) Development of reasonable and reliable concepts for seismic and tsunami-resistant design of facilities for coping with severe accidents, etc. as well as tsunami-resistant design of facilities for design basis events.

7.4.1 How to Cope with Strong Earthquakes from 2006 to 2011 and Until Now

We had experienced strong or large-scale earthquakes before and even since the establishment of the 2006 Seismic Design Review Guide, for example, the 1995 Southern Hyogo Prefecture Earthquake, the 2005 Miyagi Prefecture Oki (Miyagi Prefecture Oki: literally, off the coast of Miyagi Prefecture) Earthquake, and the 2011 Tohoku Oki Earthquake.

In the 1995 Southern Hyogo Prefecture Earthquake, seismic data in the vicinity of the epicenter were collected for the first time. From these data, valuable findings and knowledge were obtained of activity pattern of fault, dynamic characteristics of earthquake, etc. and brought rapid progress in seismology and earthquake engineering, leading to the revision of the 1981 Seismic Design Review Guide and the issue of the 2006 edition. Large-scale earthquakes continue to occur after the 1995 Hyogo Prefecture Earthquake. Meanwhile, the 2007 Niigata Chuetsu Oki Earthquake happened, causing very intensive ground motions at the site of TEPCO K-K nuclear station, which is consisting of Units 1 to 7. Maximum accelerations on the base mats of the reactor facilities exceeded their design basis earthquake ground motions (before that time, called S_2). In particular, the maximum accelerations at the Unit 1 to 4 that are located on the Arahama side (the southern side of the station) greatly exceeded S_2 . The maximum value of exceedance is approximately 2.5 at the Unit 1 [43]. At the time of this earthquake's occurrence, a geological survey was in progress near the site of station, in response to the 2006 Seismic Design Review Guide.

Almost concurrently, the following other large-scale earthquakes happened: the 2007 Noto Hanto (Noto Hanto: literally, Noto peninsula) Earthquake (M_j 6.9), whose epicenter was off the coast of Wajima City, Ishikawa Prefecture, occurred on March 20, 2007, and the 2008 Iwate-Miyagi Nairiku (Nairiku: literally, inland) Earthquake (M_j 7.2), whose epicenter was the southern inland Iwate Prefecture occurred on June 14, 2008.

And, three years later, the 2011 Tohoku Oki Earthquake happened.

Here, describe the 2007 Niigata Chuetsu Oki Earthquake because of recognizing the importance of grasping the underground structures at the site of nuclear reactor facility. Next, we discuss how to cope with this earthquake.

The 2007 Niigata Chuetsu Oki Earthquake occurred near the TEPCO K-K nuclear station. It was the first time in the world that a large-scale earthquake occurred near a nuclear power plant. Figure 7.8 shows acceleration seismograms recorded in the vicinity of the epicenter [45]. In this figure, these records are given from the top, in the north–south direction (denoted as NS), in the east–west direction (EW), and the up and down direction (UD). The maximum acceleration in the north–south direction was 667 Gal, in the east–west direction 514 Gal, in the up and down direction 369 Gal, and 3-component synthesis 812 Gal. These seismic waveforms are referred to large-scale earthquake records acquired by the K-NET system that the independent administrative agency National Research Institute for Earth Science and Disaster Prevention (NIED). The location recorded is the K-NET Kashiwazaki (NIG 018, installed on the site of the civic center in Kashiwazaki City), a few kilometers away in the southwest from the K-K nuclear station. Here, “Gal” (or “gal”) is a unit of acceleration, identical to cm/s^2 . Since 3 different pulse waves are observed, it is estimated that the seismic source fault possesses 3 different asperities (see Fig. 7.9). The maximum acceleration on the base mat of the reactor building of the Unit 1 due to this earthquake was 680 Gal, 2.5 times higher than 273 Gal, the design basis earthquake ground motion on the same base mat that was determined at the design stage. The design basis earthquake ground motion on the free surface of the base stratum was 450 Gal, common to Units 1 to 7 [43].

It was clarified from a detailed analysis of these seismograms and geological survey results that the reasons why the observed value exceeded the design basis earthquake ground motion, which was determined at the design stage, were identified as follows [43, 44]:

- (i) Unique seismic source characteristics,
- (ii) Three-dimensional irregularities in deep soil, and
- (iii) Fold structure in the vicinity of the site.

Their details are as follows:

- (i) Regarding seismic faults, three asperities were found on the sea side (the upper left figure in Fig. 7.9). Slightly larger stress drops were found than those in the other areas. Locations of these asperities are slightly different between researchers, probably due to differences in evaluation methods applied.
- (ii) There are irregularities in deep underground structure in a wide area around the site (the outline of the underground structure along the section A-A in the upper left figure in Fig. 7.9 is shown in the upper right figure). The Chuetsu region in Niigata Prefecture is covered with a deep sedimentary layer, and in the vicinity of the site, the layer boundary tends to get deeper from the land to the sea side. Particularly, changes in the horizontal direction is larger in the area close to the site than that in the west side area, resulting in layer amplification rates of seismic wave in the area close to the site than that in the west side area.

Hence, the seismic waves reaching from the sea side tend to amplify due to such three-dimensional irregular underground structure in the wide area.

- (iii) Seismic waves reaching from No.3 asperity that is located to the southwest of the site are amplified 2 to 4 times more on the Arahama side (on the side of Units 1 to 4) than those on the Oominato side (on the side of Units 5 to 7). Meanwhile, regarding seismic waves from the other two asperities and faults on the land side, there are little differences in the amplification rate between on the Arahama side and on the Oominato side. Such differences are found due to fold structures in a relatively shallow place deeper than the free surface of the base stratum. The lower right figure in Fig. 7.9 illustrates the outline of the underground structure along the section B-B (see the lower left figure) on the Arahama side where the Unit 1 is installed. The Unit 1 is located above a valley of the folds of the upper surface of the Shiiya layer and the upper surface of the upper Teradomari layer, while the Unit 5 is located above a mountain of the folds. Such a difference is a cause of larger amplification on the Arahama side than that on the Oominato side. This tendency is found to be particularly evident among earthquakes that occurred in the southwest of the site.

In the 1981 Seismic Design Review Guide, a design basis earthquake ground motion was established on the basis of results on active fault survey and others, and using the response spectrum method. The response spectrum method used the “Ohsaki’s spectrum” as the response spectrum, as mentioned earlier. The fact that the ground motions at the K-K nuclear station due to the Chuetsu Oki Earthquake remarkably exceeded the design basis earthquake ground motion may suggest that the ground motions exceed the “Ohsaki’s spectrum.” In the 2006 Seismic Design Review Guide, the response spectrum was modified from the “Ohsaki’s spectrum” to the “Seismic Design Special Committee’s spectrum,” as described in Item [6] (1) in Sect. 7.3.3. Further, the method using a fault model was newly incorporated as the evaluation method of earthquake ground motions. How the “Seismic Design Special Committee’s spectrum” improves the “Ohsaki’s spectrum” is briefly explained in Item [6] (1) in Sect. 7.3.3. The “Regulatory Guide for Safety Review on Seismic Safety of Nuclear Power Reactor Facilities” (accepted by the Nuclear Safety Commission on December 20, 2010) requires to evaluate response spectra of earthquake ground motions, taking into consideration impacts of seismic wave propagation on them due to underground structure around a site of a nuclear power reactor, based on characteristics of empirical correlations to be referred to. Also, the Regulatory Guide above requires to adequately conduct an analysis of earthquake-related seismic source characteristics due to active faults and of underground structure characteristics on the basis of geological survey data, etc. The Nuclear Safety Commission requested not only the K-K nuclear station but also the other nuclear facilities to check whether or not seismic source characteristics and underground structure characteristics are adequately taken into account in generating design basis earthquake ground motions.

Although the K-K nuclear station underwent the earthquake ground motions exceeding those assumed at the design stage, the Units 3, 4 and 7 in operation,

and Unit 2 in startup at that time were automatically and safely scrammed, and the related cooling systems were actuated and operated as designed, safely leading to the cold shutdown state. The confinement of radioactive materials was functioned as expected, and therefore, the radiation dose was far below 1×10^{-6} mSv, and extremely low comparing to the effective dose limit of 1 mSv/y, even though there were a leakage of spent fuel pool water in the Unit 6 and a release of a very small amount of radioactive materials in the Unit 7 due to a delay in stopping the operation of turbine grant steam blowers at the automatic shutdown of the reactor. Any harmful damage was not found in the facilities important to seismic safety, as explained above. However, there were several damages in the surrounding facilities with lower seismic importance as follows:

- Regarding Seismic class B facilities, failure of a drive shaft coupling of the overhead traveling crane in the reactor building of the Unit 6 (however, the fall of crane did not occur).
- Concerning Seismic class C facilities, a fire at an on-site transformer at the Unit 3, damage of outdoor piping for fire extinguishing, fall of drums in the solid waste storage facility (drum yard) and so on were reported.

TEPCO checked the facilities and conducted their structural integrity assessments promptly after the earthquake. Further, TEPCO carried out geological surveys, and reestablished the design basis earthquake ground motions. Seismic reinforcement was implemented as necessary on the basis of evaluations of the facilities using the reestablished ground motions. The maximum accelerations of the reestablished ground motions were 2300 Gal on the free surface of base stratum at the Units 1 to 4, and correspondingly, 1209 Gal at the Units 5 to 7. The Government reviewed the seismic safety assessments that TEPCO conducted using the reestablished design basis earthquake ground motions, and concluded that the seismic safety was assured [46, 47]. These reactors for which the reviews were finished resumed commercial operations from the end of 2009 to the beginning of 2011, while the Units 2, 3 and 4 were under inspection or assessment mentioned above when the Fukushima Daiichi nuclear accident occurred. The reviews by the Government were interrupted, and finally all reactors were shut down.

There may be great concern for structural integrity of the facilities and equipment, that is, whether or not stresses produced by the earthquake would greatly exceed the allowable limits, when the accelerations reach 2.5 times more than the design values or the design basis earthquake ground motions S_2 . Seismically induced stresses are not dominant in many components and parts. A dominant stress in a general part (remote from structural discontinuities) of pressure vessel shell is due mainly to internal pressure. Typical examples of components and parts where an earthquake-induced stress is dominant may be as follows:

- Parts connecting piping, which connects a vessel to another vessel or to pump, to the vessel. An earthquake-induced stress may be produced there by a bending moment that is caused by a relative gap between both vessels due to the earthquake.

- Reactor containment vessel piping penetrations and cable penetrations as parts subjected to a bending moment induced by a relative gap as mentioned above.
- Support structures.

It was found as a result of check and integrity assessment for the K-K nuclear station facilities and equipment that seismic responses of equipment of Seismic class S facilities to the 2007 Niigata Chuetsu Oki Earthquake were below the allowable limits III_{AS} (as mentioned in Sect. 7.5.2, the allowable limits III_{AS} are modified to the allowable limits for the operating condition III for structural design so as to take into account characteristics of earthquake loading, meaning that the responses remain actually within the elastic regime as gross distortion. Since the ground motions exceeding the design basis earthquake ground motions S_2 were exerted on the equipment, it may be reasonable that the allowable stress limits IV_{AS} are applied as the acceptance criterion. However, it is not easy to restart the reactor if the allowable stress limits IV_{AS} is used as the acceptance criterion, because there is the possibility of a gross plastic deformation. It is easy to restart the plant if the gross areas remain within the elastic regime, because no significant damage will be developed.

The reasons why the K-K nuclear station remained within the elastic regime against such a strong earthquake were thought to be due to the following:

- The seismic forces are not the determinant, and
- In safety regulations in Japan, as specified in the Seismic Design Review Guide, the Seismic class S facilities (or Seismic class A facilities in the 1981 Seismic Design Review Guide) are required to withstand not only dynamic seismic forces but also static seismic forces, and that the static seismic forces are calculated as three times the static seismic forces for general facilities. Rules for general buildings are stipulated in the Building Standards Act. As mentioned earlier, three times the seismic forces that are specified in the Building Standards Act are equivalent to those subjected to the acceleration of approximately 0.6 g or 600 Gal. The application of both the dynamic seismic forces and static seismic forces to the Seismic class S facilities is aimed at ensuring a certain level of seismic safety, taking into consideration a case when the dynamic seismic forces are set low. This may be because Japan is prone to earthquakes. The fact that the seismic safety of the K-K nuclear station was ensured against the 2007 Niigata Chuetsu Oki Earthquake may be said to be owing to this unique framework of seismic safety regulations in Japan.

7.4.2 The 2011 Tohoku Oki Earthquake and Tsunami

As mentioned repeatedly, the 2011 Tohoku Oki Earthquake (the 2011 off the Pacific coast of Tohoku Earthquake) of the moment magnitude M_w 9.0 caused the giant tsunami. This tsunami triggered core meltdowns of the Units 1, 2 and 3 of the TEPCO Fukushima Daiichi nuclear power station. Discharge and leakage of fission products contained in fuel claddings and hydrogen gas generated during the accident to the

outside of the reactor containment vessels caused hydrogen explosions and releases of a large amount of radioactive materials to the environment.

Mechanisms for generating the largest scale earthquake and giant tsunami ever recorded in Japan are inferred as follows:

(1) Generation of the largest scale earthquake

It is inferred that several seismic source areas covered the offshore areas of Iwate Prefecture, Miyagi Prefecture, Fukushima Prefecture and Ibaraki Prefecture (see Fig. 7.10 [48–51]), which were thought to cause large-scale earthquakes of magnitude 8 class or of magnitude 7 class individually, were interlocked. A rupture process of these seismic sources interlocked is considered to broadly consist of three stages.

The first rupture that was initiated from the seismic source beyond the offshore area of Miyagi Prefecture is continued for approximately 40 s. The epicenter position is shown in Fig. 7.10, referring to a release by the Japan Meteorological Agency.

The second rupture that was caused continuously was very large, resulting in the biggest acceleration peak. Such a fracture process can be confirmed through seismograms recorded at MYG004 (Tsukidate-Town, Kurihara-City, Miyagi Prefecture) and MYG012 (Shiogama-City, Miyagi Prefecture), both of which are the K-NET observation points managed by the Independent administrative agency National Research Institute for Earth Science and Disaster Prevention (NIED) [45], as shown in Figs. 7.11 and 7.12.

After about 100 s from the beginning of the earthquake, the rupture propagated to the southward as the third rupture (see Fig. 7.13. This seismogram was recorded at the K-NET IBR003 (Hitachi-City, Ibaraki Prefecture)) [45].

The maximum accelerations (three component synthesis) at these three observation points were 2933 Gal at MYG004, 2019 Gal at MYG012, and 1845 Gal at IBR003, respectively. These values were top 3 in acceleration data at the K-NET observation points.

Figure 7.14 shows records of UD (upper and lower) component of seismograms recorded by underground accelerometers at observation points of KiK-net (Kiban Kyosin Network) operated by the NIED, lining up in the north–south direction on the Pacific coast in the areas ranging from the Tohoku district to Ibaraki Prefecture [52]. It is found that two groups of seismic waves generated during the first and second stages of rupture process spread in the north–south direction, starting from each seismic source. Meanwhile, on the Ibaraki Prefecture side where the rupture propagated southward, the third wave group originating from the seismic wave generated during the third rupture process that occurred in the area from the Fukushima Prefecture Oki to the Ibaraki Prefecture Oki was observed, while there, those two wave groups were not clearly observed.

Furthermore, a shallow plate near the Japan Trench in the land side plate caused an exceptionally large slip at the trench. It had been well known that near a shallow plate boundary, a kind of tsunamigenic earthquake, which causes a large-scale tsunami although its shaking is not strong, rarely occurs [53]. Mechanisms of the 2011 Tohoku

Oki Earthquake might be considered that several seismic sources to cause trench type earthquakes are interlocked, with interlocking a tsunamigenic earthquake.

(2) Generation of giant tsunami

Where the oceanic Pacific Plate subducts beneath the continent side plate, the Pacific Plate is bent due to its subduction, and normal faults (see Fig. 7.5) are formed on the extension side (or tensile side) surface of the Pacific Plate. Near the Japan Trench, these normal faults produce horst and graben structures as shown in Fig. 7.15a–1 [54]. The horst and graben structure consists of alternating ridges and valleys, namely, raised and lowered fault blocks. The horst is the raised block, i.e., a fault block that has been lifted relative to the blocks on either side, and the graben is the lowered block. Figure 7.15a–1 was obtained from the seismic survey in the area of off the coast of Sanriku on the cruise KR97-07 from September to October 1997 by the Deep Sea Research Vessel “Kairei” that the Japan Agency for Marine-Earth Science and Technology (JAMSTEC) owns.

A trench type earthquake that occurred deep in the plate boundary triggered a long-wave-length tsunami. During the 2011 Tohoku Oki Earthquake, as shown in Fig. 7.15b, a large slip occurred on the shallow portion of the plate-boundary fault on the land side of the Japan Trench, probably producing the displacements of about 50 m in the horizontal direction and about 7 to 10 m upward [56]. As shown in Table 7.6, these slip values are much larger, comparing to other strong earthquakes [58–61]. The large slip produced the tsunami with a short wavelength and high wave height. The maximum run-up height of 40.1 m was recorded in Ohama City, Iwate Prefecture [62]. Note that at most tidal stations along the coast from Aomori Prefecture to Fukushima Prefecture, records on tsunami height were failed due to outflow, submergence, and power outage.

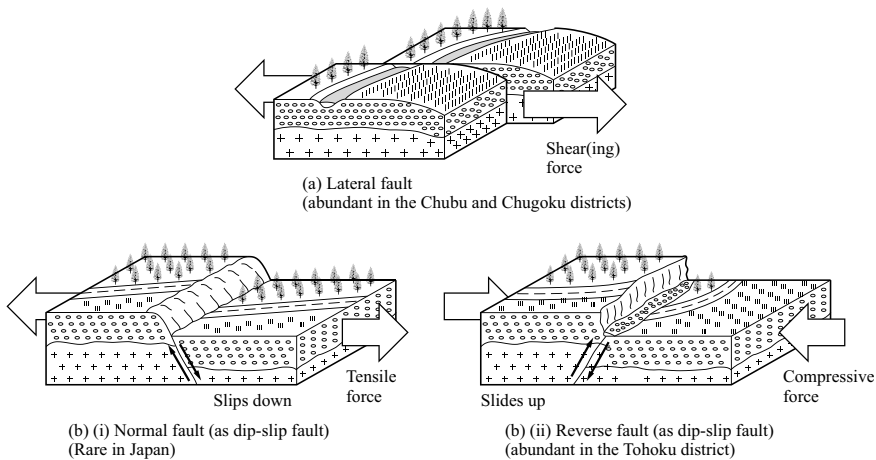


Fig. 7.5 Classification of faults

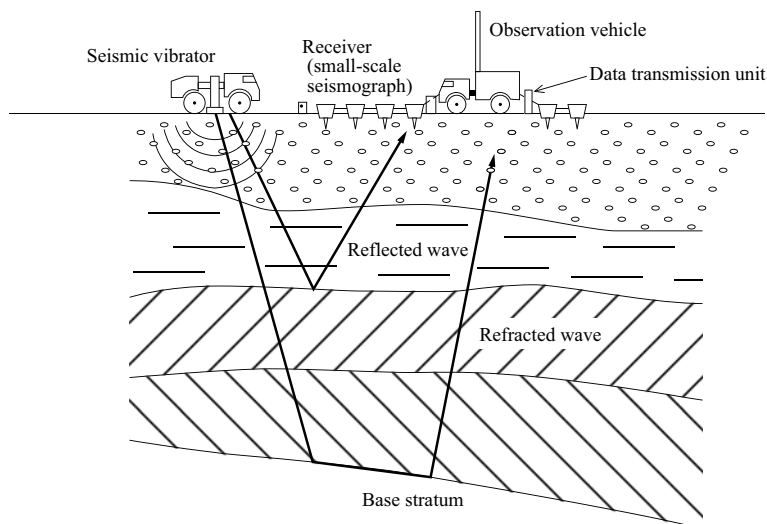


Fig. 7.6 Schematic view of land-based elastic wave reflection survey

It had been thought that the shallow portion of plate-boundary fault along the Japan Trench would not cause a coseismic slip. In order to explore causes of the large slip at the plate boundary in the shallow area near the Japan Trench, the Japan Trench Fast Drilling Project (JFAST) using the deep sea scientific drilling vessel “Chikyu” (conducted on April 1 to May 24, 2012, and on July 5 to 19, 2012) and the related researches were performed. During these expeditions using the vessel “Chikyu,” a borehole temperature observatory was installed in the Japan Trench through the plate-boundary fault zone, and temperature measurements (direct measurements of residual frictional heat caused by fault movement) and retrieval of core samples were conducted at the plate-boundary fault, soon after the 2011 Tohoku Oki Earthquake. These were technologically thought to be extremely difficult. It was clarified from an analysis of these data that in the horst-graben structure in the area of the Japan Trench shown in Fig. 7.15a-2, a large slip extended close to the trench in the very shallow (thin) fault zone under very low shear stress conditions [55, 63–65] (Fig. 7.15a-2 shows the eastern side of the seismic source (epicenter) of the 2011 Tohoku Oki Earthquake, off the coast of Miyagi Prefecture, while Fig. 7.15a-1 does off the coast of Sanriku, on the northern side of that of Miyagi Prefecture.) The plate boundary fault that the rupture extended close to the trench at the 2011 Tohoku Oki Earthquake contains the abundance of low-strength and low-permeable smectite (about 78%). Pore fluid (fluid contained in pores in plate-boundary fault substance) that was expanded due to frictional heat at the fault is thought to make the fault slippery (to reduce shear stresses) by increasing pore fluid pressure as a result of losing a refuge due to being sandwiched between low-permeability strata. Such a phenomenon is

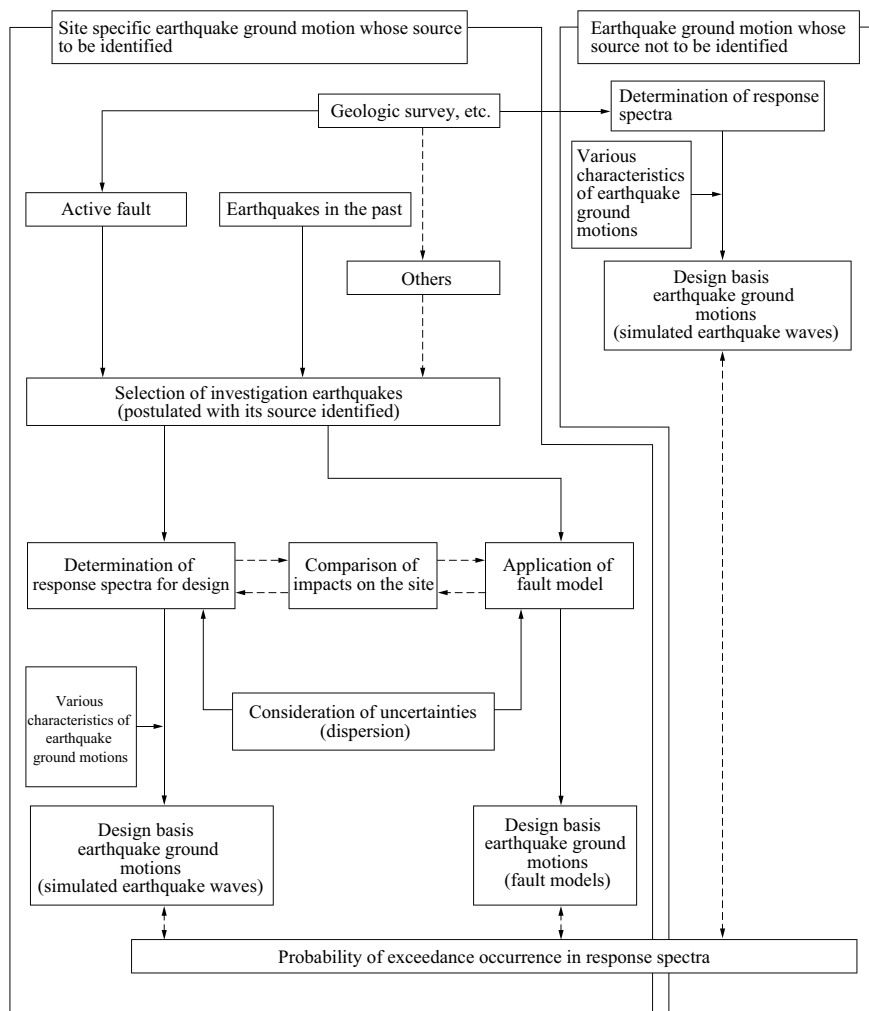
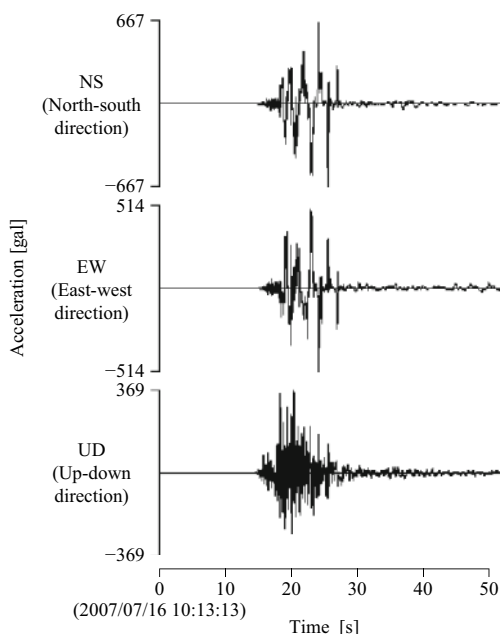


Fig. 7.7 Outline of the procedure for determining design basis earthquake ground motions [37]

called thermal pressurization. Thus, the fault was supposed to be in an extremely slippery condition at the Earthquake. The plate-boundary fault was probably only about 5 m thick, and low-strength due to inclusion of a large amount of smectite, making the fault easier to move.

Fig. 7.8 Strong ground motion waveforms of the 2007 Niigata Chuetsu Oki Earthquake [45] (Acceleration waveforms. The earthquake occurred at 10:13 AM on July 16, 2007. Measured at the observation point K-NET Kashiwazaki (NIG 018) of the National Research Institute for Earth Science and Disaster Prevention)



7.4.3 *How Did the Strong Earthquake and Tsunami Change Regulations on Seismic and Tsunami-Resistant Design?*

The Nuclear Safety Commission (NSC) established the Subcommittee on Reviewing Guidelines relating to Earthquake and Tsunami in the Special Committee on Nuclear Safety Standards and Guidelines, for discussing on how to reflect lessons and findings from the 2011 Tohoku Oki Earthquake and the subsequent tsunami, and also the Fukushima Daiichi nuclear accident. On March 14, 2012, the subcommittee released a report entitled “Issues for reflecting to the Seismic Design Review Guide and Related Guides (summary)” that summarized investigations on the causes to have the accident as a result of failing to predict the earthquake and tsunami. Key issues pointed out in the report are as follows:

- In the 2006 Seismic Design Review Guide, a tsunami was treated as an earthquake-accompanying event. From investigations on tsunami cases happened over the world and tsunami generation mechanisms, however, it can be said that main causes for generating tsunamis are not only tsunamigenic earthquakes but also volcanic mountain collapses and landslides. Hence, a new chapter on tsunami-resistant design should be set as separated from the chapter on seismic design, and rules necessary to evaluate tsunami should be included in the new chapter.
- Regarding the evaluation of earthquake and earthquake ground motion, the following were pointed out.

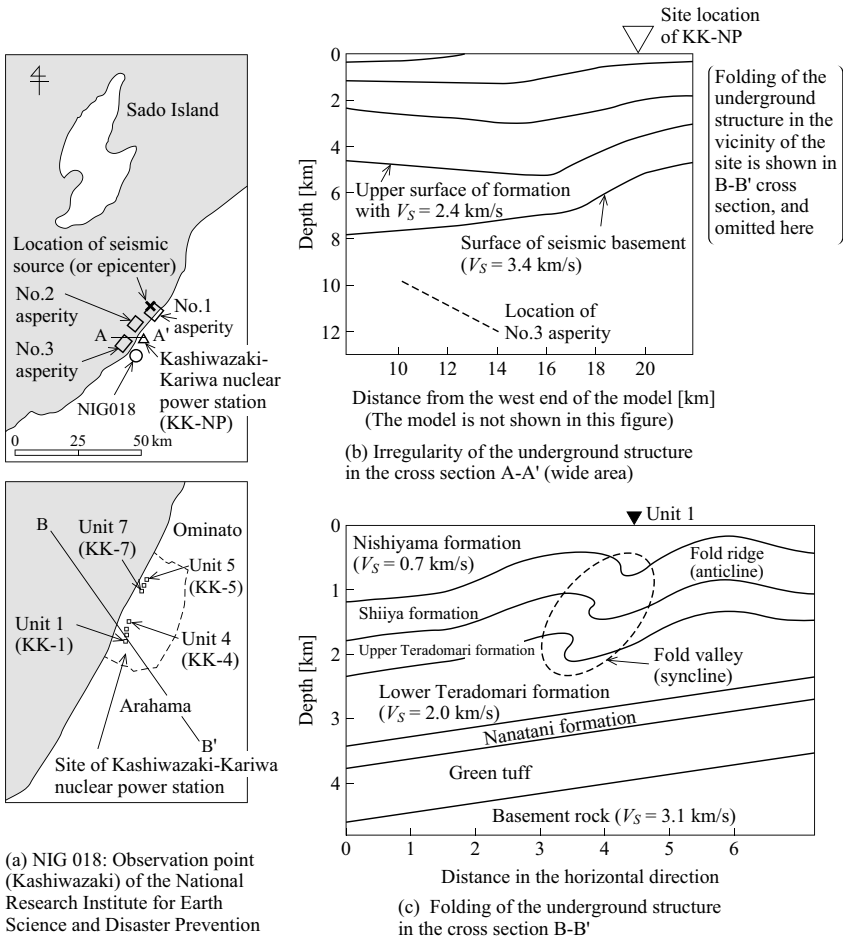
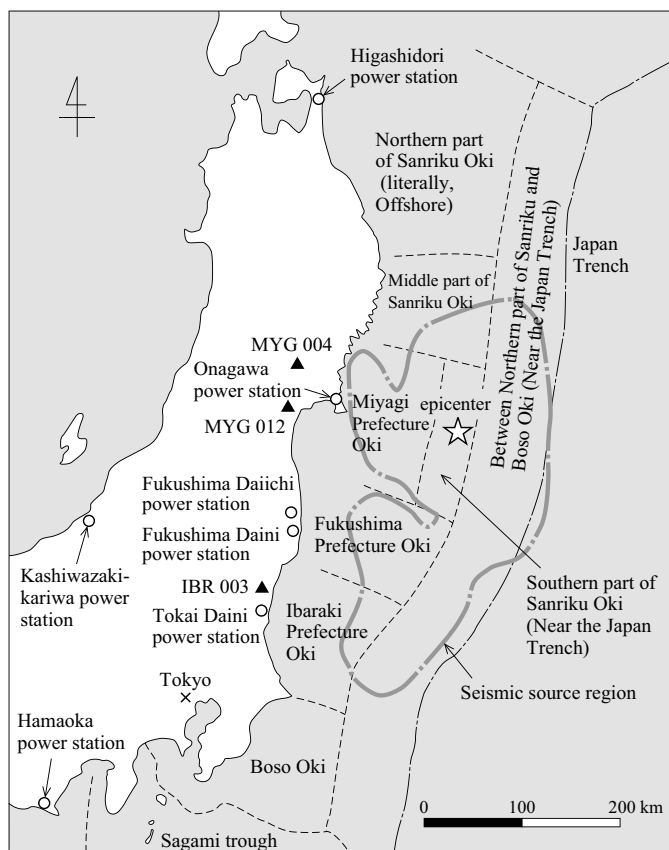


Fig. 7.9 Asperities to cause the 2007 Niigata Chuetsu Oki Earthquake, and irregularity and folding that are estimated as the underground structure in the vicinity of the site of Kashiwazaki-Kariwa nuclear power station [43, 44]

- (i) The 2006 Seismic Design Review Guide mainly provided rules for evaluating inland crustal earthquakes. Rules for evaluating uncertainty (or scatterings) about fault area, earthquake scale, etc. also should be given on interpolate earthquakes and ocean intraplate earthquakes.
- (ii) Taking it into consideration that no matter whether there is a past case and no matter where the earthquake occurs, a megathrust earthquake cannot be denied to occur at a boundary of subducting plate, world cases of megathrust earthquakes that occurred at plate boundaries and that had similar mechanisms for



▲ : Locations of the K-NET observation points shown in Figures 7.11 to 7.13
 ○ : Locations of nuclear power stations

[Note 1] The earthquake occurrence area classification map that had been established at the time of the 2011 off the Pacific coast of Tohoku Earthquake was referred to as the classification map shown above (this map has been revised in 2019).

[Note 2] Here, the postulated seismic source region was derived from the contour line with slip amount of 4 m that was computed using a coseismic slip distribution model developed by the Geospatial Information Authority of Japan (through March 2010, it was called the Geographical Survey Institute)⁵⁰⁾.

(Shimazaki proposed to define the seismic source region as the contour line of 4 m slip⁵¹⁾.)

Fig. 7.10 Earthquake occurrence area classification of trench type earthquakes (or subduction zone earthquakes) that were occurred off the Pacific coast of Japan (the Headquarters for Earthquake Research Promotion, Ref. [48]), the location of mainshock of the 2011 off the Pacific coast of Tohoku Earthquake (the Japan Meteorological Agency), and the postulated seismic region (the Geospatial Information Authority of Japan, Ref. [50])

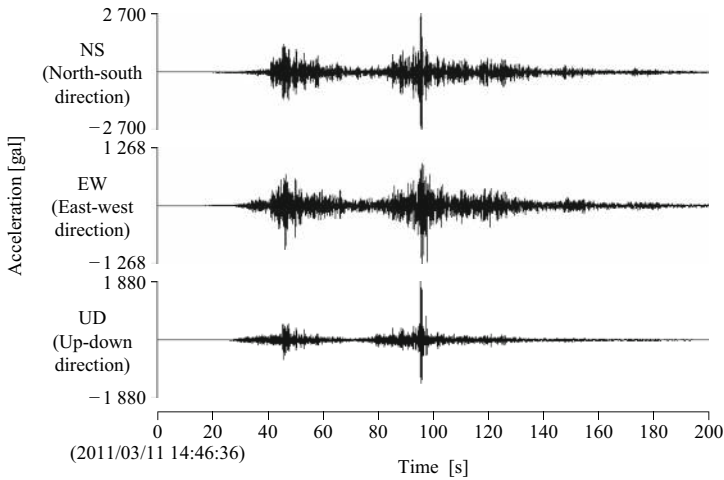


Fig. 7.11 Acceleration waveforms recorded at the observation point MYG 004 (Tsukidate Town, Kurihara City, Miyagi Prefecture) of K-NET [45]

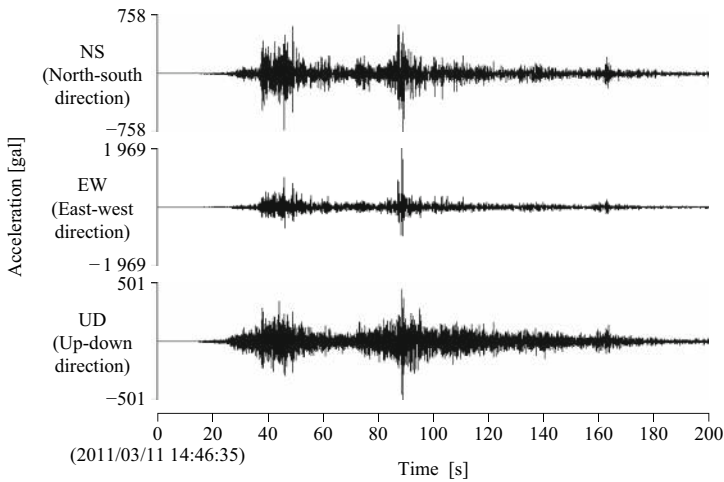


Fig. 7.12 Acceleration waveforms recorded at the observation point MYG 012 (Shiogama City, Miyagi Prefecture) of K-NET [45]

generating earthquakes and tectonic backgrounds should be considered in evaluating earthquake ground motions for establishing a design basis earthquake ground motion at a nuclear reactor site.

- (iii) From a viewpoint that a crustal movement, an important earthquake phenomenon, should be taken into account in assessing tsunami and ground stability, it is required to describe as a basic policy of a guideline for reviewing

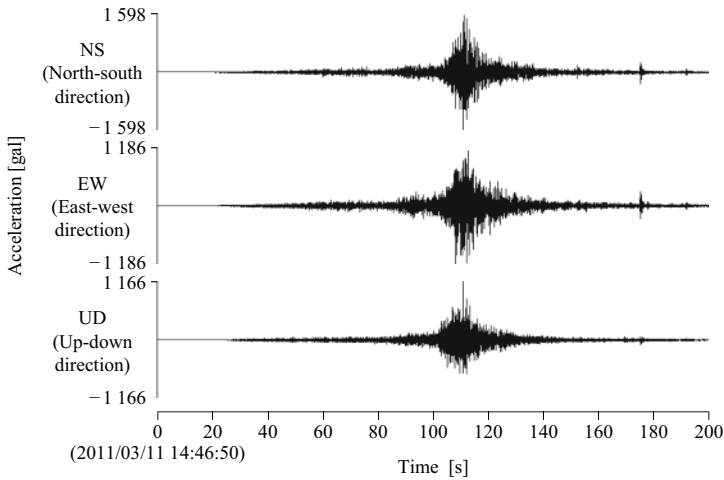


Fig. 7.13 Acceleration waveforms recorded at the observation point IBR 003 (Hitachi City, Ibaraki Prefecture) of K-NET [45]

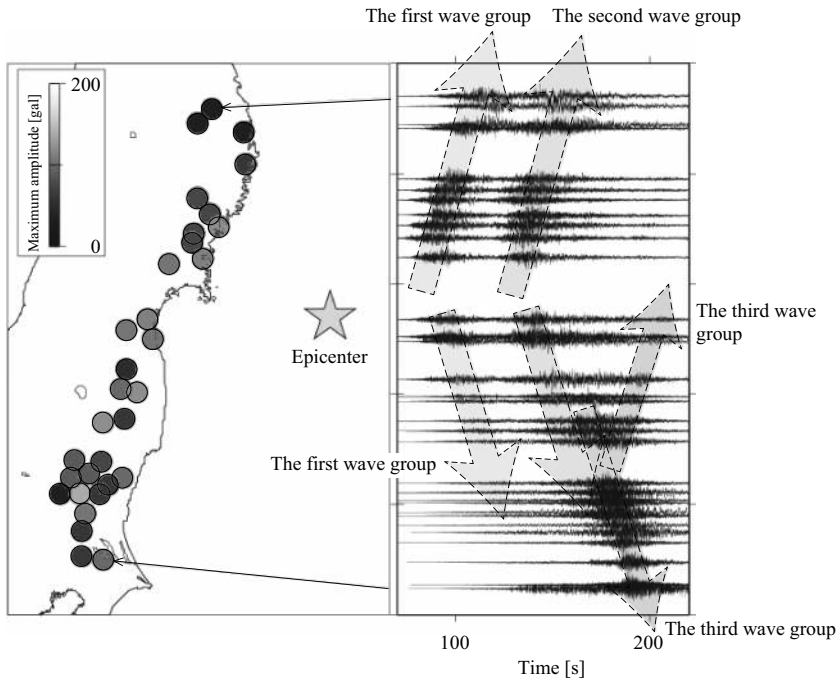


Fig. 7.14 Features of earthquake waveforms recorded on the Pacific coast of Tohoku district (Earthquake waveforms recorded at KiK-net of the National Research Institute for Earth Science and Disaster Prevention)

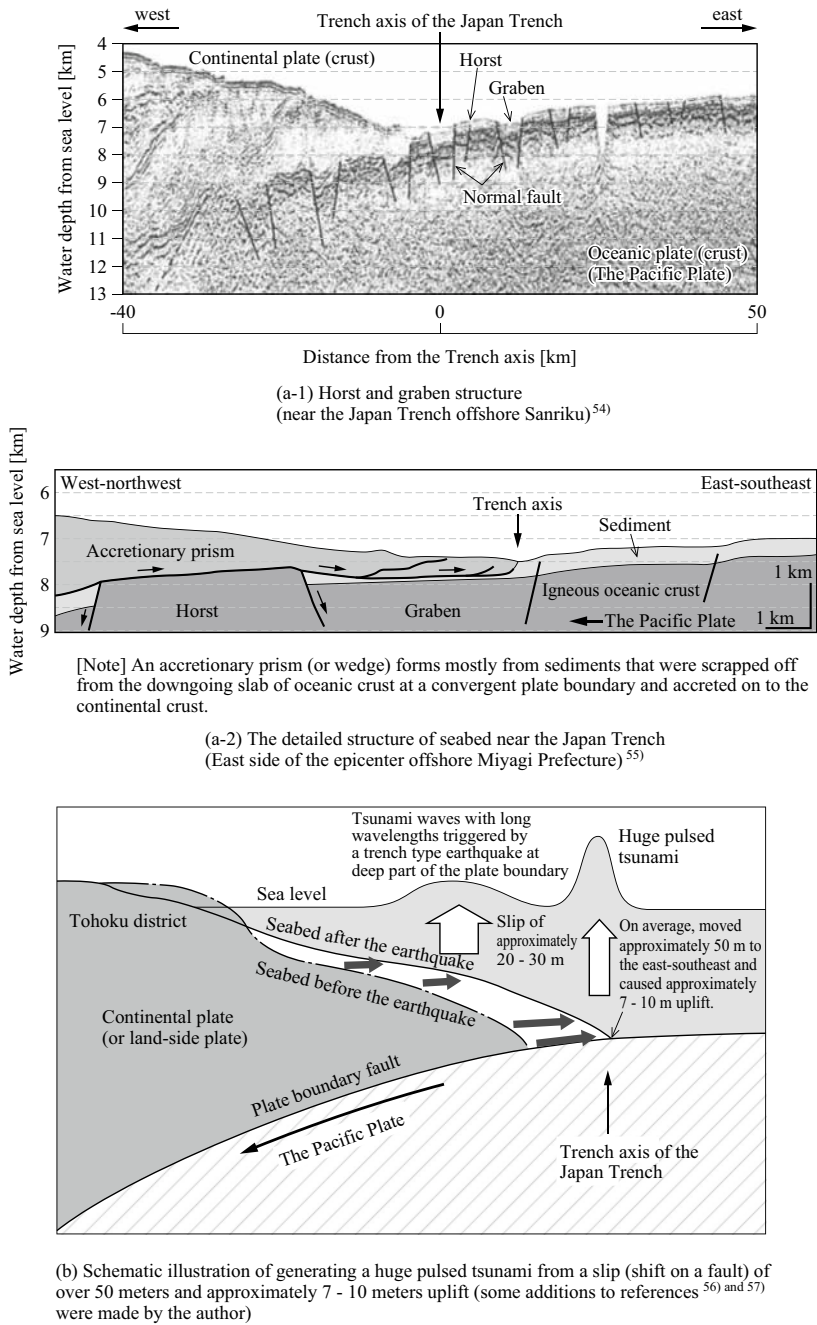


Fig. 7.15 Seabed structure near the Japan Trench and mechanisms of generating huge tsunamis triggered by the 2011 off the Pacific coast of Tohoku Earthquake [54–57]

Table 7.6 Seismic source (epicenter) data of large-scale earthquakes over the world [58]

Earthquake	Intensity (moment magnitude M_w)	Length and width of seismic source area	Mean amount of slip	Maximum amount of slip
1960 Chilean Earthquake	9.5	Approx. 1 000 km × Approx. 200 km	11 m	25 – 30 m ⁵⁹⁾
1964 Prince William Sound (off the Coast of Alaska) Earthquake	9.2	Approx. 500 km × Approx. 300 km	9 m	22 m ⁶⁰⁾
2004 off the Coast of Sumatra Earthquake	9.1	Approx. 1 000 km × Approx. 150 km	9 m	23 m ⁶¹⁾
2011 off the Coast of Tohoku Earthquake	9.0	Approx. 400 km × Approx. 200 km	10 m	Over 50 m

a seismic and tsunami-resistant design that a crustal movement shall be clearly investigated and that items relating to an evaluation of support performance of the ground should be provided.

- (iv) From a viewpoint that it is necessary to assume a wide variety of earthquakes after a huge earthquake occurs, an investigation on occurrence of a different generation pattern of earthquake due to stress propagation caused by the huge earthquake (for example, occurrence of an inland crustal earthquake triggered by a huge interpolate earthquake) should be conducted in such a way that the possibility of its occurrence should be conducted on the basis of scientific findings and that an assessment needs to be implemented based on these investigation results.
 - (v) There is uncertainty (scattering) in determining active faults to be evaluated for a seismic design, even though surveys as required are fully completed. In determining the active faults, the judgment should be made, taking into account the uncertainty.
 - (vi) It was mentioned in the “Regulatory Guide for Safety Review on Seismic Safety of Nuclear Power Reactor Facilities” that in determining active faults for the seismic design, if terrain surfaces and strata of Late Pleistocene are not distributed, further older age terrain geology, geological structure, stress field and so forth should be investigated comprehensively, and then they should be determined. Based on findings related to the 2011 Tohoku Oki Earthquake and the subsequently triggered earthquakes, crustal movements, and so on, we should consider the possibility that the stress fields are drastically changed due to earthquakes occurred in the vicinity.
- Regarding the tsunami evaluation,
 - (i) A tsunami is required to be formulated as the basis of safety design of nuclear reactor facilities. This design basis tsunami should be established as the tsunami appropriate to have likelihood to extremely rarely occur during the service life, and to have the possibility of heavily affecting the facilities.

- (ii) On the basis of the latest findings, an appropriate level of uncertainty (scattering) should be taken into consideration in establishing the design basis tsunami. While the design basis earthquake ground motion is established on the basis of active fault surveys and so forth, it is stipulated in the Seismic Design Review Guide and the above-mentioned Regulatory Guide that the design basis tsunami should be established as that affects heavily the facilities, by establishing such a wave source to cause the largest scale tsunami on the basis of tsunami generation mechanisms, because surveys of tsunami marks, etc. in the vicinity of the site of the nuclear reactor facilities are not enough to establish the largest scale tsunami. In this regard, tsunami sources should be basically established by taking into consideration similarity of generation mechanisms and tectonic backgrounds, referring to domestic and global cases on tsunami. Further, it should be confirmed that in the surrounding of the site, there is no trace of such a tsunami that exceeds the tsunami height predicted from the tsunami wave source as established.
 - (iii) The principle of safety assessment against tsunami is to prevent the invasion of the design basis tsunami into the site by means of installing protective facilities. Flooding into the site through openings of water intake facilities and water discharge facilities that are functionally difficult to prevent flooding seawater is allowed only for areas where safety functions of the facilities are not damaged by flooding of tsunami.
 - (iv) We cannot deny the possibility of striking the nuclear power plant by such a tsunami exceeding the design basis tsunami that is established with scientific rationality on the basis of the latest findings. Even in this situation, measures should be taken to suppress any influence on the public in the surrounding of the site as a result of radiation-induced disaster.
- Regarding issues and opinions to be resolved in future
 - (i) Facilities and activities for assuring safety of a nuclear power plant should be constantly reviewed while reflecting the latest findings and knowledge. From now on, a Review Guide on Earthquake and Tsunami to be established and the related guides should be applied to existing nuclear power plants as well as the newly constructed plants (so-called “back-fit”).
 - (ii) It is important to revise the guide on seismic design importance classification and to newly stipulate rules on importance classification of facilities and equipment related to tsunami-resistant design. Since the loss of off-site power due to the 2011 Tohoku Oki Earthquake and tsunami is one of critical causes of the Fukushima Daiichi nuclear accident, it is important to take drastic measures for appropriately ensuring the seismic safety of off-site power receiving facilities and so forth.
 - (iii) An emergency information system, which is prepared so as to tailor to plant-specific features, will become feasible by constructing their own networks without relying on emergency information to be issued by the Japan Meteorological Agency. It is important for the nuclear reactor operator (or licensee)

himself/herself to build observation networks relating to earthquakes and tsunamis, and to establish frameworks required.

- (iv) Hereafter, it is necessary to carry out investigations on volcanos, etc. as other natural phenomena than earthquakes and tsunamis.

The NRA that took over the NSC set up the “Review Team on Countermeasures for Light Water Type of Nuclear Power Reactor Facilities against Earthquakes and Tsunamis” and the team implemented reviews of the related regulatory standards. The review result was recognized to be as important as a basis of reviewing regulatory standards. Reflecting this review, regulatory standards were revised. For instance, the “Ordinance on Standards for Location, Structures and Equipment of Commercial Nuclear Power Reactors and their Auxiliary Facilities” (as mentioned above, referred to as the “Ordinance on Standards for Location, Structures and Equipment” or “Establishment Permit Standards”) (the NRA Ordinance No. 5, June 28, 2013) and the “Interpretation (or Regulatory Guide) of the Ordinance on Standards for Location, Structures and Equipment of Commercial Nuclear Power Reactors and their Auxiliary Facilities” were established, in such a way that concerning the part related to earthquake and tsunami, the “2006 Seismic Design Review Guide” was drastically changed, reflecting the review result. In addition, the following Regulatory Guides and others were developed, reflecting the establishment of the Ordinance and its Interpretation:

- the “Regulatory Guide on Design Basis Earthquake Ground Motions and Seismic Design Guideline,”
- the “Regulatory Guide on Design Basis Tsunami and Tsunami-Resistant Design Guideline,” and
- the “Regulatory Guide on Stability Assessment of Ground and Peripheral Slope.”

Major changes of the “2006 Seismic Design Review Guide” are as follows [65]:

- (1) Seismic design importance classification of measures (or facilities, etc.) against tsunami and earthquake

The 2011 Tohoku Oki Earthquake attacked nuclear power stations that were located on the Pacific side of eastern Japan. The following nuclear power stations were subjected to the maximum earthquake ground motions at the level of design basis earthquake ground motions:

- The Onagawa Nuclear Power Station, owned by the Tohoku-EPCO (Tohoku Electric Power Co., Inc.)
- The Fukushima Daiichi and Daini (Daiichi and Daini: literally, No.1 and No. 2) Nuclear Power Stations, owned by the TEPCO
- The Tokai Daini (literally, No.2) Nuclear Power Station, owned by the JAPC (Japan Atomic Power Company)

Subsequently, huge tsunamis attacked these stations, where the maximum heights of tsunamis were at the level of design basis tsunami or much higher.

Taking into account these facts, facilities having functions to protect the reactor against tsunami, such as a seawall, are required to be classified as the “facilities for design basis events,” and to withstand the design basis earthquake ground motions, with maintaining the functions. Further, these facilities should perform tsunami-resisting functions against the design basis tsunamis generated by the earthquakes. Consequently, these facilities should be classified as Seismic class S.

The following facilities should be included as Seismic class S in addition to the facilities classified as the Seismic class S facilities in the 2006 Seismic Design Review Guide:

- Facilities and equipment having functions to protect the reactor against tsunamis (they are called the “tsunami protection facilities.” And those having functions to prevent flooding (they are called the “flooding prevention equipment.”). The “tsunami protection facilities” include a seawall for protecting in-flow of tsunami, equipment for protecting in-flow through an intake channel to a pump area. The “flooding prevention equipment” includes equipment for preventing influence on safety functions by means of restricting flooded areas, and watertight doors for preventing in-flow into the inside of the facility. Facilities and equipment having tsunami monitoring functions on the site (called the “tsunami monitoring equipment”). These equipment includes water level gauges at an intake pit, and outdoor monitoring cameras.

(The above-mentioned is referred to the pages 100 and 103 of the NRA document entitled “On concepts of new regulatory standards with regard to commercial nuclear power reactors” [66].)

- (2) Seismic and tsunami resistance of facilities for coping with severe accidents, etc.

The facilities for coping with severe accidents, etc. are required to have no possibility of damaging functions necessary to cope with severe accidents, etc. under natural conditions (earthquake, tsunami, and others), internal fires, and social conditions (intentional crash of a large aircraft, etc.). Therefore, against an earthquake and tsunami,

- The ground on which the facilities concerned are installed should have an enough capability to support them, in order to prevent any damage of functions to cope with severe accidents, etc. due to serious influence on supporting functions of the buildings and so on, by applying seismic forces developed by the design basis earthquake ground motions.
- There is no possibility of damaging any functions required due to applying seismic forces developed by the design basis earthquake ground motions.
- There is no possibility of damaging any functions required due to the design basis tsunamis.

(The above-mentioned is referred to the pages 140 and 141 of Ref. [66].)

- (3) Specialized safety facilities among facilities for coping with severe accidents, etc. These facilities are required to withstand seismic forces equivalent to those which are applied to the Seismic class S facilities among the facilities for design basis events, with a sufficient margin, and to maintain functions necessary to cope with severe accidents, etc. for seismic forces due to the design basis earthquake ground motions. Concerning seismic resistance against the design basis earthquake ground motions, it is required to enhance robustness against earthquake ground motions that exceed the design basis earthquake ground motions to a certain extent, by taking a diversity of measures, namely, taking measures different in character from those for design basis events. An example as diversity measures is a seismic isolation structure or seismic control (or damping) structure.

Design allowable limits for the design basis tsunamis are the same as those for facilities for design basis events.

Further, it is required to enhance robustness against tsunamis that exceed the design basis tsunamis to a certain extent, by taking a diversity of measures, namely, taking measures different in character from those for design basis events. Examples are storage of the equipment concerned in buildings with guaranteed water tightness or in buildings that are installed on a hill.

(The above-mentioned is referred to the page 142 of Ref. [66].)

- (4) Multi-layered measures against tsunamis, taken for facilities for design basis events

There is the possibility that a tsunami exceeding the design basis tsunami, which is established with scientific rationality on the basis of the latest knowledge, would attack a nuclear power plant. Taking into consideration the possibility, multi-layered tsunami measures should be taken. The multi-layered measures are based on a concept of “drysite,” namely, to protect the site from running up or causing an inflow. Further, a certain level of resistance is required for events beyond a design basis (for example, an event at which a tsunami goes over a seawall and causes an inflow to the site).

The detail of the multi-layered measures is as follows:

- (i) Outer protection 1;

Against the design basis tsunami, it is required to prevent flooding or an inflow of run-up waves from the ground to the site where the Seismic class S facilities are installed, and an inflow through channels such as an intake channel and drainage channel. If there is the possibility of an inflow of the tsunami through channels such as an intake channel and drainage channel, measures should be taken for preventing flooding.

(ii) Outer protection 2;

If there is the possibility of leakage in an intake and drainage facility, underground facility, etc. a range of flooded area as a result of continuation of leakage should be restricted, and measures against flooding should be taken. If there is the Seismic class S equipment in the vicinity of the estimated flooded area, the area should be designed as a water-tight compartment. Drainage equipment should be installed if a long time flooding is expected in the estimated flooded area, as a result of assessment of flooded water amount and of impacts on facilities, equipment, etc. with safety functions.

(iii) Inner protection;

Buildings and compartments that contain the Seismic class S equipment should be clearly classified as prioritized areas for preventing flooding, and measures for preventing flooding should be taken for channels and ports with the possibility of flooding into the area concerned.

(The above-mentioned is referred to pages 305 and 306 of Ref. [66].)

(5) Beyond the assumptions for facilities for coping with severe accidents, etc.

As mentioned above, facilities for coping with severe accidents, etc. are designed to prevent any damage of functions that are required to cope with severe accidents, etc. against the design basis earthquake ground motions and design basis tsunamis. Basically, measures against severe accidents, etc. are not designed to cope with natural phenomena whose scales are beyond assumptions made in the design. If facilities for coping with severe accidents, etc. that have functions to prevent failure of a reactor containment vessel would be designed to withstand earthquake ground motions and tsunamis exceeding the design basis earthquake ground motions and design basis tsunamis, respectively, there might be the possibility of the containment vessel failure due to such earthquake ground motions and tsunamis, because the containment vessel is designed as a facility for design basis events. Such design requirements are not thought to be reasonable.

Further, a facility does not always fail under the conditions of earthquake ground motions and tsunamis even a little over the design basis earthquake ground motions and design basis tsunamis, respectively, as always in design.

Measures against severe accidents, etc. are taken in combination with portable equipment. Since portable equipment can flexibly cope with severe accidents, etc., it may be said that a high level of reliability is assured against earthquakes and tsunamis as a whole of facilities and equipment involved as measures against severe accidents, etc.

When as an unlikely event, such a large-scale natural disaster as to exceed the one caused by the natural phenomenon assumed as the design occurs, and consequently a large-scale collapse of the nuclear power plant facility is caused, measures against the large-scale collapse are separately required for mitigating impacts due to severe damage of the reactor core and failure of the containment vessel, and so forth.

(The above-mentioned is referred to the pages 212 and 214 of Ref. [66].)

- (6) Exceedance probability of design basis earthquake ground motions and residual risk.

In generating design basis earthquake ground motions due to inland crustal earthquakes, for example, it is impossible to predict in advance all earthquake-related processes such as a fault rupture process. For other reasons as well, there is the possibility of earthquake ground motions exceeding the design basis earthquake ground motions. In addition, it is not required to deny occurrence of such earthquake ground motions. Accordingly, it is required for licensees (or business operators) to refer to exceedance probabilities of the specified design basis earthquake ground motions. In new regulatory standards, which the NRA has been developing, the technical term “residual risk” that was defined in the 2006 Seismic Design Review Guide disappeared. However, it is thought that the licensees continue efforts to minimize the residual risk as a result of always recognizing the possibility of occurrence of earthquake ground motions with strength exceeding the design basis earthquake ground motions by referring to the exceedance probability of the design basis earthquake ground motion, and then of giving adequate considerations in designing facilities.

(The above-mentioned is referred to the page 291 of Ref. [66].)

7.4.4 Regulations for Facilities for Design Basis Events and Those for Coping with Severe Accidents, Etc.

As mentioned above, new regulatory requirements corresponding to the previous 2006 Seismic Design Review Guide (but drastically changed) are provided in the following Ordinance and Regulatory Guide, and others, as a result of reviews by the review subcommittee on regulatory guides regarding earthquakes and tsunamis, and by the review team for countermeasures in a light water reactor type of nuclear power plant facilities against earthquakes and tsunamis:

- the “Ordinance on Standards for Location, Structures and Equipment of Commercial Power Reactors and their Auxiliary Facilities” (the Nuclear Regulation Authority Ordinance No. 5, 2013. Here, the revised version issued on December 14, 2017 is referred to. In this Sect. 7.4.4, this is called the “Ordinance.”)
- the Interpretation (or Regulatory Guide) of the “Ordinance” (enacted on June 19, 2013. Here, the revised version issued on March 13, 2019 is referred to. In this Section, this is called the “Interpretation.”).

How the review group changed the draft prepared by the review subcommittee are summarized as “points of issues” which were discussed at each meeting.

In the Ordinance and its Interpretation, key items are given in the following Articles:

For facilities for design basis events;

- requirements on ground: Article 3 of the Ordinance,

- requirements on how to prevent damage due to earthquake: Article 4,
- requirements on how to prevent damage due to tsunami: Article 5, and
- The detailed requirements by revising the 2006 Seismic Design Review Guide: the Separate Notes 1, 2, 3 and 4 of its Interpretation.

For facilities for coping with severe accidents, etc.;

- requirements on ground: Article 38 of the Ordinance,
- requirements on how to prevent damage due to earthquake: Article 39, and
- requirements on how to prevent damage due to tsunami: Article 40.

The main parts of the 2006 Seismic Design Review Guide (in the following, simply denoted as the 2006 Guide) are referred to Article 4 of the Ordinance, because the 2006 Guide stipulated requirements for facilities for design basis events. Requirements on support performance of the ground were stipulated with other seismic design requirements, mainly requirements on earthquake, in the 2006 Guide, but in the new regulatory standards, or the Ordinance, they are independently stipulated in the separate Article 3. Requirements on tsunami are also stipulated in the separate Article 5. The Separate Notes of its Interpretation incorporate guidelines that were provided as the Explanation (or Interpretation) of the 2006 Guide into their main texts. Furthermore, the contents and requirements themselves are changed a lot from those of the 2006 Guide. The following, therefore, gives the full text of the Ordinance and its Interpretation.

The definitions of key technical terms that are newly used in the Ordinance and its Interpretation are as follows:

- Combinedly usable cask; the cask is usable as conforming to regulations relating to transportation of nuclear fuel materials, etc. and can be combinedly usable as a container that is utilized for transporting to the outside such as a spent fuel factory, among dry casks for storing spent fuels in the factory, etc.
- Facilities for coping with severe accidents, etc.; the facility has functions to cope with not only beyond design basis accidents with the possibility of escalating to a severe accident but not severe accident itself (excluding anticipated operational occurrences and design basis accidents) but also severe accidents themselves. Integration of both categories of beyond design basis accidents is called “severe accidents, etc.”
- Specialized safety facilities; the facility is functioned to suppress a release of an abnormal level of radioactive materials to the outside of the factory, etc. due to failure of the reactor containment vessel at a beyond design basis accident event with the possibility of escalation to causing a severe damage of the reactor core or due to severe core damage by an intentional aircraft crash and other terrorism actions. This facility belongs to facilities for coping with severe accidents, etc.
- Equipment for coping with severe accidents, etc.; the equipment has functions for coping with severe accidents, etc.
- Equipment for preventing severe accidents; the equipment has functions to prevent occurrence of a severe accident by means of substituting (or recovering) a lost

function (limited to a function necessary to cope with beyond design basis accidents with the possibility of escalating to severe accidents) at an event of losing safety function(s) of equipment for coping with design basis accidents or a cooling function of spent fuel storage tanks, or a water injection function.

- Equipment for mitigating severe accidents; the equipment has functions to prevent escalation of the severe accident concerned, or to mitigate their impacts at a severe accident event. It is included in equipment for coping with severe accidents, etc.

In the following, [commentary] by one of the authors is added to the Ordinance and Interpretation, as needed.

[1] Rules for ground, prevention of damage due to earthquake and that due to tsunami for facilities for design basis events

[Ground for facilities for design basis events]

Rules of the Ordinance

Article 3

Subarticle 1

The facilities for design basis events should be established on the ground with a sufficient support capacity for the facilities for design basis events concerned against seismic forces that are calculated according to rules specified in the next Article, Subarticle 2 (for such facilities as to make remarkably large radiation impacts on public that are triggered by a loss of safety functions which is assumed to be caused by an earthquake (hereinafter, referred to as the “seismically important facilities” or facilities important to seismic safety), the seismic forces include those due to the design basis earthquake ground motions to be established in accordance with rules specified in the same Article (Article 4), Subarticle 3).

[Author’s commentary].

The seismically important facilities are identical to the Seismic class S facilities of facilities for design basis events.

Subarticle 2

Seismically important facilities and combinedly usable casks should be established on the ground where there is no possibility of losing safety functions that they have even though they are deformed.

Subarticle 3

Seismically important facilities and combinedly usable casks should be established on the ground with no possibility of deformation. If the combinedly usable casks are capable of being established by means of such a method that there is no possibility

of losing the safety functions even though any ground deformation is induced, the above requirement need not apply.

[Commentary].

The implication of “seismically important facilities and combinedly usable casks should be established on the ground with no possibility of displacement,” specified in this Subarticle 3 is as follows in regard to the seismically important facilities. This is given in the Separate Note No.1 of the Interpretation.

[Separate Note No. 1]

If seismically important facilities would be established on the ground with a outcrop of a fault, etc. that has the possibility of becoming active in future, there would be the possibility of damaging safety functions. Consequently, these facilities should be established on the ground that is confirmed no outcrop of a fault, etc. with the possibility of becoming active in future. The clause “a fault, etc. with the possibility of becoming active in the future” means that its activity later than the Late Pleistocene (later than $1.2\text{--}1.3 \times 10^5$ years ago) cannot be denied. In determining its activity, if the activity later than the Late Pleistocene is not clear, for instance, due to lack of terrain surface or strata of the Late Pleistocene, the activity should be investigated going back to the Middle Pleistocene (later than 4.0×10^5 years ago) on the basis of an integrated consideration of geography, geology and geological structure, stress field, etc. If in investigating the activity, it is difficult to check it at the area for establishing the facilities, it should be judged on the safe side, referring to properties, etc. of the fault, etc. that were found in the extension of the fault concerned. The fault, etc. with the possibility of becoming active in future includes an active fault to be taken into account as a seismic source (or hypocenter), a fault to cause permanent deformations as a result of seismic activities, and a landslide surface whose displacements and deformations extend to the supporting ground.

Rules for combinedly usable casks and the surrounding facilities (hereinafter called “combinedly usable cask storage facilities”) are provided in the Separate Note No. 4. In this subsection, the Separate Note No. 4 is omitted.

[Prevention of Damages Due to Earthquakes]

Rules of the Ordinance

Article 4

Subarticle 1

Facilities for design basis events should withstand seismic forces with a sufficient margin.

Subarticle 2

Seismic forces specified in the preceding subarticle should be calculated by grading the levels of radiation-induced impacts on public that attribute to loss of safety functions of facilities for design basis events that may be triggered by an earthquake.

Subarticle 3

Seismically important facilities should have no possibility of damaging safety functions against seismic forces induced by accelerations due to the earthquake that has the possibility of damaging the seismically important facilities concerned during their service (hereinafter called “seismic forces due to the design basis earthquake ground motions”).

Subarticle 4

Seismically important facilities should have no possibility of damaging safety functions against slope collapse that may be caused due to the earthquake specified in the preceding subarticle.

[Commentary].

In the 2006 Seismic Design Review Guide, the rules of subarticle 4 were specified as epiphenomena induced by an earthquake.

Subarticle 5

Fuel claddings in a reactor core should have no possibility of damaging functions to contain radioactive materials against seismic forces due to the design basis earthquake ground motions.

Subarticle 6

Combinedly usable casks should have no possibility of damaging safety functions against the following seismic forces:

1. The seismic forces specified by the Nuclear Regulation Authority as reasonable seismic forces for use in judging whether or not to damage safety functions against seismic forces.
2. Seismic forces due to the design basis earthquake ground motions.

Subarticle 7

Combinedly usable casks should have no possibility of damaging safety functions against slope collapse that may be caused by an earthquake.

Rules of the Interpretation

(they are provided as the Separate Note No.2 as described below. Here, the Separate Note No. 4 is omitted, which provides an interpretation on containment functions of fuel claddings in a reactor core, and of combinedly usable cask storage facilities.)

[Separate Note No. 2]

1. The clause “to withstand seismic forces with a sufficient margin” mentioned in Subarticle 1, Article 4, means that major parts of the facility concerned as a whole facility should be designed within an elastic regime against the specified seismic forces. In this meaning, “designed within an elastic regime” means that stresses in each part of the facility should be less than or equal to allowable limits. The allowable limits are not necessarily based on an elastic limit but can be determined so as to keep the stresses in the major parts as a whole facility within the elastic regime. This allows the stresses to locally exceed the elastic limit.

[Commentary].

The above elastic design concept of keeping the stresses in the major parts within the elastic regime is equivalent to that for allowable limits of the operating condition III or service condition C, which are specified for designing structural components as discussed in Chap. 5. A requirement for the elastic design concept may be that stresses in major parts other than those that are necessary to repair, for instance, because of leakage at structural discontinuities, etc. should be restricted in the elastic regime where no damage is allowed, as explained by “shutdown for repair of the part concerned” or “allowing large deformations at structural discontinuities.”

Such stress conditions that large deformations are permitted to develop even in general parts, or in plain parts far away from structural discontinuities, are allowed only for the operating condition IV or service condition D. Components with safety functions that should be performed at an accident should be designed within a limited deformation range so as to safely terminate the plant at the accident.

Regarding examination, after an event categorized as the operating condition III or service condition C, only the part concerned is required to be examined before the restart of plant, while after an event of the operating condition IV or service condition D, examination should be made over the entire areas.

It may be noted here that static seismic forces and dynamic seismic forces established from the elastic design earthquake ground motions S_d are applied to the facility as the design specification or design conditions for sizing bar arrangements, etc. of buildings, namely, for their elastic design.

2. As speculated in Subarticle 2 in Article 4, “the levels of radiation-induced impacts on public that attribute to loss of safety functions of facilities for design basis events” mean gross levels of impacts at an event of loss of safety functions of each facility (herein after called “importance in seismic design”) from viewpoints of preventing loss of safety functions of facilities for design basis events, which may occur due to an earthquake (including loss of safety functions due to a tsunami, collapse of peripheral slope, etc., which may occur following the earthquake), and of preventing the subsequent radiation-induced impacts on public. Facilities for design basis events are classified into the classes shown in Table 7.7 in accordance with the importance in seismic design.
3. In order to satisfy the requirement “to withstand seismic forces with a sufficient margin,” which is specified in Subarticle 1 in Article 4, seismic designs of the

Table 7.7 Seismic importance classification of facilities for design basis events

Seismic importance class	Functions	Facilities
S	The following safety functions and facilities to achieve their functions at an event that may be caused by an earthquake, and their influence (to be caused by loss of their safety functions) are significant. The facilities given in the right hand column should be included. (i) Facilities to have functions necessary to cool the reactor core, (ii) Facilities to contain radioactive materials in themselves, (iii) Facilities directly related to the item (ii) above, and to have the possibility of discharging radioactive materials to the outside if their functions are lost, (iv) Facilities to have functions necessary to mitigate impacts when loss of functions of the facilities mentioned above causes an accident, and to reduce radiation-induced consequences to the public, (v) Facilities necessary to support those important safety functions, and (vi) Facilities necessary to prevent loss of safety functions due to tsunami that has the possibility to be caused by an earthquake.	a. Components and piping systems composing of the “reactor coolant pressure boundary” b. Facilities for storing spent fuel c. Facilities to add negative reactivity for emergency shutdown of the nuclear reactor, and facilities to preserve the shutdown mode of the nuclear reactor d. Facilities to remove decay heat from the reactor core after the shutdown of the nuclear reactor e. Facilities to remove decay heat from the reactor core after the accident of failure of the reactor coolant pressure boundary f. Facilities to directly prevent the discharge of radioactive materials directly as the pressure barrier at the failure accident of the reactor coolant pressure boundary g. Facilities to mitigate the diffusion of radioactive materials to the outside at such accident as to release them, other than the “facilities to directly prevent the discharge of radioactive materials” mentioned in the item f. above h. Equipment to have functions to protect from tsunami hereinafter, referred to as the “equipment for protecting from tsunami”), and equipment to have functions to prevent flooding (hereinafter, referred to as the equipment for preventing flooding”) i. Equipment to have functions to monitor tsunami on the site (hereinafter, referred to as the “equipment for monitoring tsunami”)
B	Facilities to have safety functions, and their influence to be caused by loss of the safety functions is less significant than that of Seismic class S facilities above. The facilities given in the right hand column should be included.	a. Facilities directly connected to the reactor coolant pressure boundary, and containing the primary coolant or with the possibility of containing it b. Facilities storing radioactive waste (excluding the facilities wherein the radiation-induced consequences resulting from their failure to the general public are sufficiently small on account of its small inventory or of the type of storage system, as compared with the annual exposure limit at the outside of the “peripheral observation area”, which is specified in Item 6, Subarticle 2, Article 2 of the “Ordinance on Installation, Operation, etc. of Commercial Nuclear Power Reactors” (the Ordinance No. 77 of MITI (Ministry of International Trade and Industry), 1978)). c. Facilities related to radioactive materials other than radioactive waste, which may potentially cause, as a result of their failure, excessive radiological exposure to the general public and the operational personnel d. Facilities to cool spent fuel e. Facilities to mitigate the diffusion of radioactive materials to the outside at such event as to release them, other than Seismic class S facilities
C	Facilities other than Seismic class S or B facilities that are required to ensure the safety level equivalent to general industry facilities or public facilities.	Facilities other than the above-mentioned Seismic class S or B facilities.

facilities belonging to each class of classification of importance in seismic design should be conducted in accordance with the following guideline.

(1) Seismic class S facilities (except facilities for protecting from tsunami, equipment for protecting from flooding, and equipment for monitoring tsunami)

- To withstand the greater of seismic forces due to the elastic design earthquake ground motions or static seismic forces in the range of keeping stresses in the elastic regime in major parts
- For buildings and structures, loads acting at any time and those acting during operation are combined with seismic forces due to the elastic design earthquake ground motions or static seismic forces. The allowable unit stresses (or stress degrees) that are tabulated in such codes and standards that are recognized to be suitable from a view point of safety, for example, the Building Standards Act, should be applied as allowable limits for the stresses resulting from the combined loads and forces.
- For components and piping systems, their responses should be kept in the elastic regime in major parts as a whole for combined loading conditions of loads to be generated at normal operations, anticipated operational occurrences and accidents, respectively, and seismic forces due to the elastic design earthquake ground motions or static seismic forces. With regard to the respective loads generated during anticipated operational occurrences and accidents, loads emerging from an event that may potentially be caused by an earthquake, and those due to an event that lasts long once it occurs although its occurrence due to an earthquake may not be probable should be considered in combination with adequate seismic forces in view of relationship of occurrence probability of the event, its duration and exceedance probability of the earthquake ground motion.

(2) Seismic class B facilities

- To withstand static seismic forces in the range of keeping stresses in the elastic regime in major parts. If there is the possibility of resonance, its impact should be evaluated. In doing so, the earthquake ground motion to be applied should be obtained by multiplying the elastic design ground motion by $1/2$.
- For buildings and structures, loads acting at any time and those acting during operation are combined with static seismic forces. The allowable unit stresses (or stress degrees) that are tabulated in such codes and standards that are recognized to be suitable from a view point of safety, for example, the Building Standards Act, should be applied as allowable limits for the stresses resulting from the combined loads and forces.
- For components and piping systems, their responses should be kept in the elastic regime in major parts as a whole for combined loading conditions of loads to be generated at normal operations and anticipated operational occurrences, and static seismic forces.

(3) Seismic class C facilities

- To withstand static seismic forces in the range of keeping stresses in the elastic regime in major parts.
 - For buildings and structures, loads acting at any time and those acting during operation are combined with static seismic forces. The allowable unit stresses (or stress degrees) that are tabulated in such codes and standards that are recognized to be suitable from a view point of safety, for example, the Building Standards Act, should be applied as allowable limits for the stresses resulting from the combined loads and forces.
 - For components and piping systems, their responses should be kept in the elastic regime in major parts as a whole for combined loading conditions of loads to be generated at normal operations and anticipated operational occurrences, and static seismic forces.
4. “Calculation” of “seismic forces” as stipulated in Subarticle 2 in Article 4 should be implemented in accordance with the following procedure.

(1) Seismic forces due to the elastic design earthquake ground motion

- The elastic design earthquake ground motion should be established from an engineering judgment so that a value of ratio of response spectra to the design basis earthquake ground motion (earthquake ground motion due to earthquakes that have the possibility of significantly affecting seismically important facilities during the service should not be lower than 0.5 as a guide. The above-mentioned definition of the design basis earthquake ground motion is adopted also in the following.
- Seismic forces due to the elastic design earthquake ground motion should be formulated as a proper combination of those in two horizontal directions and in a vertical direction. Interactions between buildings/structures and earthquakes, embedding effects and nonlinearities of the surrounding grounds should be taken into account as needed.
- In formulating seismic forces, an adequate analytical method should be selected through assessing applicability of seismic response analytical methods, their application limitations, etc., and analytical conditions should be properly established on the basis of sufficient surveys.
- For developing input ground motions that are formulated at locations to install buildings and structures in the formulation process of earthquake ground motions, propagation properties of seismic waves from the free surface of base stratum should be appropriately investigated, and dynamic deformation properties relating to nonlinear responses of the ground should be considered as needed. In addition, their adequacy should be demonstrated on the basis of observation records on the site and the latest scientific and technological findings.

(2) Static seismic force

(a) Buildings and structures

- Horizontal seismic forces should be calculated by multiplying the earthquake layer shearing force coefficient C by the following factor in accordance with classification of importance in seismic design of facilities, and further by the total weight of the layer concerned and higher.

Seismic class S 3.0

Seismic class B 1.5

Seismic class C 1.0

where the earthquake layer shearing force coefficient C_i should be calculated through setting the standard earthquake layer shearing force coefficient C_0 to 0.2 or greater, and taking into considerations vibration properties of the buildings and structures, type of the ground, etc.

- Horizontal load-carrying capacity of the buildings and structures should be confirmed to exceed the requisite horizontal load-carrying capacity.
- In formulating the requisite horizontal load-carrying capacity, the factor in accordance with the classification of importance in seismic design, by which the earthquake layer shearing force coefficient is multiplied, should be commonly equal to 1.0. And, the standard earthquake layer shearing force coefficient should be 1.0 or greater. In formulating these parameters, an adequate safety margin should be included, depending upon the seismic importance of the respective facility.
- For Seismic class S facilities, horizontal and vertical seismic forces should be combined so as to simultaneously apply in their adverse directions. The vertical seismic forces should be calculated assuming the seismic intensity of 0.3 or more as the base, and on the basis of vertical seismic intensity that is estimated by considering the vibration properties of buildings and structures, type of ground, etc. The vertical seismic intensity should be constant in the height direction.

(b) Components and piping systems

- Seismic forces for each classification class of importance in seismic design should be calculated from seismic intensities that are estimated, increasing by 20% from the horizontal seismic intensity concerned and vertical seismic intensity specified in Item (a) above. The horizontal seismic intensity concerned is derived by multiplying the earthquake layer shearing force coefficient C_i by the factor depending upon the facility classification class of importance in seismic design.
- Horizontal and vertical seismic forces should be combined so as to simultaneously apply in their adverse directions. It should be noted here that the assumption of the standard earthquake layer shearing force coefficient C_0 as 0.2 or greater is aimed at encouraging an owner of nuclear power reactor to construct facilities

with a high level of earthquake resistance, by means of assigning appropriate values to individual buildings and structures, and components and piping systems on the basis of appropriate evaluations of their importance in designing them, respectively. What amplification factor is appropriate to assign to each facility from a viewpoint of enhancing the earthquake resistance should be determined by personnel involved in design and construction by considering relationships to seismic standards for industrial and public facilities, etc.

[Commentary].

Facilities for protecting from tsunami, equipment for preventing flooding and equipment for monitoring tsunami as tsunami-resistant facilities, among the Seismic class S facilities, should maintain the functions required to these facilities and equipment against loads due to the design basis earthquake ground motions and loads at an event to be combined with the design basis earthquake ground motions. However, the requirement of elastic design, namely, to withstand those loads is not imposed. The reason may be because the design conditions should be established so as to maintain the functions required against the design basis tsunamis, primarily tsunamis that are triggered by the design basis earthquake ground motions, and therefore, so as to maintain the functions required against the design basis earthquake ground motions that would occur just before the tsunamis.

5. The “design basis earthquake ground motions” specified in Subarticle 3 in Article 4 should be established as appropriate to assume their occurrences from viewpoints of seismology on geology and geological structure, ground structure, seismicity, etc. of the site and its surrounding areas, and earthquake engineering on the basis of the latest scientific and technological findings, in accordance with the following procedure.
 - (1) The design basis earthquake ground motions should be formulated as earthquake ground motions in two horizontal directions and in a vertical direction on the free surface of base stratum, regarding the “site specific earthquake ground motions whose sources to be identified” and the “earthquake ground motions whose sources no to be identified,” respectively. The “free surface of base stratum” refers to the free surface that is virtually set up, assuming that there exist no surface layer and structures on the surface of base stratum, and the surface of base stratum with an almost horizontal and considerable extent, and with no significant elevation difference, in order to formulate the design basis earthquake ground motions. Here, the “base stratum” is defined as a hard ground whose shear wave velocity V_s is approximately 700 m/s or higher and that is not remarkably weathered.
 - (2) The “site specific earthquake ground motions whose sources to be identified” (hereinafter, referred to as EGMs with seismic sources identified at each site) should be developed as follows:
 - (1) Select such multiple earthquakes as expecting to have a great impact on the site, from inland crustal earthquakes, interpolate earthquakes and oceanic intraplate earthquakes. The earthquakes selected are called “investigation earthquakes.”

- (2) Formulate earthquake ground motions from each investigation earthquake selected, by means of both the response spectra-based method and the method using fault models, in considerations of their uncertainty and reflecting propagation properties of seismic waves up to the free surface of base stratum.

The “inland crustal earthquake” mentioned above (or the “inside island earth’s crustal earthquake”) refers to an earthquake occurring in the upper crustal seismogenic layer of a continental side plate, and includes an earthquake occurring somewhat offshore. The “interpolate earthquake” refers to an earthquake occurring at the interfacial plane where two adjacent plates contact. The “oceanic intraplate earthquake” (or the “inside oceanic plate earthquake”) refers to an earthquake occurring in the interior of a subducting or subducted oceanic plate. It is broadly classified into two groups: one is an “earthquake within a subducting oceanic plate” that occurs in the vicinity of trench axis or somewhat offshore from there, and the other is an “earthquake within a subducted oceanic plate (so-called inside slab (or intraslab) earthquake)” that occurs in an area on the continent side of the vicinity of trench axis.

[Commentary].

These definitions are mentioned in Item [4] in Sect. 7.3.3.

The above-mentioned “EGMs with seismic sources identified at each site” should be generated according to the following detailed procedure, based on the guideline mentioned above:

- (a) Select multiple “investigation earthquakes” on the basis of careful investigations on active fault properties and earthquake occurrence situations, and comprehensive reviews of past research results, etc. on distributions of medium, small and micro earthquakes, stress fields, and seismogenic modes (including plate geometries, movements and interactions) for inland crustal earthquakes, interpolate earthquakes and oceanic intraplate earthquakes.
- (b) Regarding inland crustal earthquakes, the following should be taken into consideration:
 - (i) Evaluation of an active fault to be considered as a hypocenter (or a seismic source) should be made so as to clarify its location, geometry, activity, etc. by comprehensively assessing results of surveys implemented by means of properly combining various methods such as survey of existing literature, geomorphological survey, geological survey and geophysical survey with effectively taking advantage of features of each survey method.
 - (ii) In evaluating geometries of seismic source models, parameters of seismic source properties, etc., careful judgment on isolated and short active faults should be made, and interlocking ruptures of multiple active faults should be considered.
- (c) Regarding interpolate earthquakes and oceanic intraplate earthquakes, the seismic source region should be determined in consideration of a similarity of seismogenic mechanisms and tectonics backgrounds, referring to large-scale earthquakes that occurred in the world as well as in Japan.

- (d) The earthquake ground motions should be generated using both the response spectrum-based method to be specified in Item (i) below and the method by means of fault models to be specified in Item (ii) below, for respective investigation earthquakes selected in Item (a) above. In generating the earthquake ground motions, various properties (including a size effect) depending upon seismogenic modes, propagation paths of seismic waves, etc. should be fully considered based on observation records at the site.
 - (i) Response spectrum-based method for generating earthquake ground motions

The earthquake ground motions should be generated by establishing “design response spectrum” as a result of evaluating response spectrum using an adequate evaluation method for respective investigation earthquakes, and then for the “design response spectrum,” by taking into proper consideration features of the earthquake ground motions such as durations of the earthquake ground motions and aging effects of amplitude envelopes on the basis of scale of earthquake, distance from seismic source, etc.
 - (ii) Earthquake ground motion generation by applying methods using fault models

Earthquake ground motions should be generated by selecting proper fault models, and then, by establishing seismic source property parameters of the fault models selected, for respective investigation earthquakes.
- (e) Regarding various kinds of uncertainty in process of generating the earthquake ground motions (uncertainty in length of seismic source fault, top and bottom depths of seismogenic layer, dip of seismic source, location and size of asperity, stress drop amount, rupture initiation point, and uncertainty caused by differences in approach and interpretation associated therewith), governing parameters to be considered to have significant effects on generation of the earthquake ground motions at the site should be analyzed, and then their uncertainty should be taken into account using a proper method, for instance, by combining their uncertainty as needed.
- (f) If among the investigation earthquakes selected from inland crustal earthquakes, the seismic source of the earthquake is extremely close to the site, it is required to investigate in detail adequacy of geometry and location of seismic source model and validity of locational relationships with the site and with facilities installed on the site, and of seismic source parameter setting, with covering the entire fault that is accompanied by a displacement on the surface. Based on the investigation results, attention should be paid to applicability of the evaluation methods, and an evaluation should be made in more detail of effects that the uncertainty revealed in Item (e) above have on generation of the earthquake ground motions. Thus, the design basis earthquake ground motions should be established with sufficient margins on the basis of the latest scientific and technological knowledge relating to features of earthquake ground motions that have been observed or generated extremely close to seismic sources.

- (g) Surveys and evaluations to be made for selection of investigation earthquakes and establishment of design basis earthquake ground motions should be based on the latest scientific and technological knowledge and findings. Further, past documents, etc. should be referred to, in consideration of their sufficiency and accuracy. In addition, if a view or approach different from that cited in the past documents is adopted or a result different from the past evaluation results is obtained, its basis should be clearly shown.
 - (h) If a seismic isolation facility is adopted, and more generally, a relatively long period component is predominant in its seismic response, an earthquake ground motion evaluation should be conducted with paying attention to its frequency characteristics. It is required to establish a separate design basis earthquake ground motion for this facility from those for the other facilities on the identical site, as needed.
- (3) The “earthquake ground motions whose sources not to be identified” (hereinafter, referred to as EDMs without seismic sources identified) should be established based on response spectra that are generated by reflecting the ground characteristics of the site in consideration of various kinds of uncertainty, referring to measurement records in the vicinity of seismic sources, which were collected about past inland crustal earthquakes that are difficult to correlate seismic sources with active faults. The “EGMs without seismic sources identified” should be established in accordance with the following procedure:
- (a) It is required to suitably reflect propagation properties of seismic wave up to the free surface of base stratum to generate response spectra, as needed. Further, it is required to take into proper consideration features of the earthquake ground motions such as durations of the earthquake ground motions and aging effects of amplitude envelopes for the generated response spectra.
 - (b) Adequacy of the “EGMs without seismic sources identified” should be individually confirmed on the basis of the latest scientific and technological findings at the time of application of establishment permit, etc. In establishing the earthquake ground motions, it is required to refer to evaluations by taking into account various kinds of uncertainty such as a probabilistic evaluation, about earthquake ground motions in the vicinity of seismic sources triggered by seismic source faults that do not leave their traces on the ground surfaces.

[Commentary].

Standards response spectra to establish the “EGMs without seismic sources identified” [67].

The “EGMs without seismic sources identified” refers to earthquake ground motions to commonly incorporate at all sites (or target sites) regardless of results of detailed surveys in the vicinity of the site, because even if detailed surveys are conducted by sufficiently investigating situations in the surrounding of the site, it is impossible to say definitely that all inland crustal earthquakes with the

possibility of occurring in the surrounding of the site are evaluated in advance. Two causes of such earthquakes are known:

- (a) Earthquakes to be caused nationwide (approximately M_w 6.5 or less)
An earthquake having the possibility of not emerging as a surface earthquake fault, and to be considered to occur anywhere in Japan, without identifying location and scale of the seismic source in advance.
- (b) Earthquakes to reflect regionality (approximately M_w 6.5 or greater)
An earthquake having the possibility of occurring in the region where the existence of an active fault was not pointed out in advance, and having found only a part of trace near the ground surface.

The Nuclear Regulation Authority (NRA) is investigating how to generate a “standard response spectrum” on a surface equivalent to a surface of so-called seismic basement for establishing the “EDGs without seismic sources identified” by limiting to earthquakes to be caused nationwide as given in Item (a) above, and then its investigation result was open to the public in August 2019 (the definition of the term “seismic basement” is provided in Item [1] in Sect. 7.5.1. Licensees (or reactor owners) will be recommended to reflect the above-mentioned result to the establishment of the design basis earthquake ground motions on the free surface of base stratum, reflecting the ground features in shallower than the seismic basement at the individual site.

- (4) Concerning surveys to be conducted for establishing the design basis earthquake ground motions, reliability and accuracy of survey results should be ensured by selecting proper survey methods to achieve its objective, and by considering application conditions, accuracy, etc. of the survey methods.

For evaluations on earthquake ground motions in establishing the “EDGs with seismic sources identified at each site” and the “EDGs without seismic sources identified,” the following matters relating to the propagation properties of seismic waves should be taken into consideration, paying attention to the property data necessary to construct evaluation methodologies to be applied:

- (a) In order to investigate effects of underground structure (deep- and shallow-level ground structure) on the propagation properties of seismic waves, it is required to assess geological structures in and around the site such as dip of stratum, fault and fold structure, and further to assess the location and geometry of seismic basement, the heterogeneity of rock facies and quality, and the underground structure such as seismic wave velocity structure and damping properties of ground. In the assessment process, the three-dimensional underground structure should be assessed, except for a case that the underground structure has been approved to be stratified and homogeneous.
- (b) Surveys in and around the site necessary to make the assessment mentioned in (a) above should be performed according to adequate procedures and combinations

of survey of regional characteristics and past literature, collection and analysis of past data, analysis of earthquake measurement records, geological survey, boring survey, 2- or 3-dimensional geophysical exploration, etc.

It is required to refer to exceedance probabilities of the “EDGs with seismic sources identified at each site” and the “EDGs without seismic sources identified,” and it should be grasped to what level of exceedance probability the response spectrum for each earthquake ground motion corresponds.

6. The facilities for design basis events should be designed in accordance with the following policies against the design basis earthquake ground motions in order to satisfy the requirement “should have no possibility of damaging safety functions” specified in Subarticle 3 in Article 4:

- (1) Seismically important facilities except those mentioned in (2) below

- To maintain safety functions against seismic forces due to the design basis earthquake ground motions
- For buildings and structures, normal loads (i.e., loads acting always) and operating loads (i.e., loads acting during operation) are combined with seismic forces due to the design basis earthquake ground motions. Against the combined loads and seismic forces, the relevant buildings and structures should maintain deformation capability (deformation at ultimate strength) as a whole structure with a sufficient margin, and possess the ultimate strength with a proper safety margin.
- Components and piping systems should be designed to maintain functions required for the facility against combined loading conditions of loads acting at normal operations, anticipated operational occurrences and accidents, and seismic forces due to the design basis earthquake ground motions. Even if plastic strains are produced upon application of the loads specified above, the amount of plastic strains is limited to such a small level as to meet a rupture ductility limit with a sufficient margin, and then without damaging the functions required for the facility. Active components should be designed to retain functions required to the equipment for responses due to the design basis earthquake ground motions. In particular, the relevant allowable limits should be established as accelerations for retaining functions, etc., which have been confirmed by verification tests.

With regard to “loads acting at anticipated operational occurrences and accidents,” loads emerging at an event that may potentially be caused by an earthquake, and loads acting at an event that lasts long once it occurs even though the event would not be caused by an earthquake should be evaluated in combination with adequate seismic forces, taking into account a relationship of occurrence probability of the event, its duration and exceedance probability of the earthquake ground motion.

- (2) Buildings and structures where facilities for protecting from tsunami, equipment for preventing flooding and equipment for monitoring tsunami are installed

- Individual facility and equipment should be designed to maintain functions required (i.e., functions to protect from tsunami, functions to prevent flooding or functions to monitor tsunami) against seismic forces due to the design basis earthquake ground motions.
- The relevant buildings and structures where facilities for protecting from tsunami and equipment for preventing flooding are installed are required to ensure the deformation capability (the deformation at the ultimate strength) as a whole structure with a sufficient margin, and to maintain the functions required to the facility (functions to protect from tsunami and to prevent flooding), against the combined loadings of normal and operating loads, and seismic forces due to the design basis earthquake ground motions.
- Equipment for preventing flooding and equipment for monitoring tsunami should maintain functions required to the facility (functions to prevent flooding and to monitor tsunami) against the combined loadings of normal and operating loads, and seismic forces due to the design basis earthquake ground motions.
- Regarding these loading conditions, the possibility of simultaneous application of earthquake and tsunami should be investigated, and a combination of seismic forces due to the design basis earthquake ground motions and loads due to tsunami should be taken into account, as needed.

The “ultimate strength” (or ultimate load-carrying capacity) is defined as a limiting maximum applied load corresponding to an ultimate situation of a structure at which deformations or strains of the structure remarkably increase as monotonically increasing the load acting on the structure.

Seismically important facilities should be designed not to lose their safety functions due to spreading impacts caused by facilities belonging to a lower class of seismic importance classification. In an evaluation of the spreading impacts, it is required to show adequacy of event selection results and impact evaluation, including the contents, etc. of surveys and investigations overlooking the entire site, and to apply the earthquake ground motions or seismic forces to be used in the design of seismically important facilities.

The above sentence “seismically important facilities should be designed not to lose their safety functions due to spreading impacts caused by facilities belonging to a lower class of seismic importance classification” means to confirm no impact on the safety functions of seismically important facilities at least regarding to the following items:

- impacts due to relative displacements or differential settlements arising from differences, etc. in the installation ground and seismic response characteristics
- mutual impacts at a connection between a seismically important facility and a facility belonging to a lower class of seismic importance classification
- impacts on seismically important facilities caused by a damage, tipover, drop, etc. of a facility of a lower seismic class inside a building
- impacts on seismically important facilities caused by a damage, tipover, drop, etc., of a facility of a lower seismic class outside a building

7. “Seismic forces due to the design basis earthquake ground motions” specified in Subarticle 3 in Article 4 should be calculated in accordance with the following procedure:

- Seismic forces due to the design basis earthquake ground motions should be calculated as appropriately combining seismic forces in two horizontal directions and in a vertical direction, which are derived using the design basis earthquake ground motions. It is required to take into consideration interactions between buildings/structures and the ground, embedding effects and nonlinearities of the surrounding ground, as needed.
 - In calculating seismic forces due to the design basis earthquake ground motions, an adequate calculation method should be selected by evaluating applicability of analytical methods for calculating seismic responses, limitations on applications, etc., and analytical conditions should be properly established on the basis of sufficient surveys.
 - For developing input ground motions that are calculated at locations, etc. to install buildings and structures in the process of calculating earthquake ground motions, it is required to approximately evaluate propagation properties of seismic waves from the free surface of base stratum and dynamic deformation properties relating to nonlinear response of the ground, as needed. In addition, their adequacy should be demonstrated on the basis of observation records at the site and the latest scientific and technological findings.
8. Subarticle 4 in Article 4 stipulates that it should be confirmed by a stability analysis of a slope surrounding a seismically important facility, subjected to seismic forces due to the design basis earthquake ground motions that the slope has no possibility of collapse, and that if there is the possibility of collapse, it is required to take measures to remove the relevant portions and to retain the slope by means of civil engineering structures in the site area, in order to prevent any impact on seismically important facilities.

The stability analysis should be conducted in accordance with the following policies:

- (1) The stability assessment should be made on a slope that has the possibility of affecting buildings containing equipment possessing functions important to safety, and outdoor equipment with functions important to safety, etc.
- (2) The assessment should be made using sliding safety factors, etc., taking into consideration structures of geology and ground, ground grade classification, liquefaction possibility, groundwater impacts, etc.
- (3) It is required to establish a ground model, ground parameters and seismic forces for use in the assessment according to an assessment of support capacity of the foundation ground.

[Prevention of Damages Due to Tsunamis]

Rules of the Ordinance

Article 5

Subarticle 1

Facilities for design basis events (excluding combinedly usable casks and their surrounding facilities) should have no possibility of damaging safety functions against tsunamis that may have a great impact on the relevant facilities for design basis events during their service (hereinafter called the “design basis tsunami”).

Subarticle 2

Combinedly usable casks and their surrounding facilities should have no possibility of damaging safety functions against any of the following tsunamis:

- (1) Tsunami that the NRA specifies as a tsunami to be reasonable for use in judging if safety functions of combinedly usable casks are damaged by it, irrespective of their locations of installation.
- (2) Design basis tsunamis

Rules of the Interpretation

(they are provided as the Separate Note No.3. Here, the Separate Note No. 4 is omitted, which provides an interpretation of combinedly usable cask storage facilities.)

[Separate Note No. 3]

1. The design basis tsunamis explained in this Article 5 should be established as appropriate to generate them from seismological points of view of submarine topography, geological structure, seismic activity, etc. in the region ranging from the wave source sea area to the site periphery on the basis of the latest scientific and technological findings. Multiple causes to generate tsunami should be selected from landslide, slope collapse and other causes than earthquake as well as earthquake, and their combinations. The design basis tsunamis should be established by conducting numerical analyses for the selected causes, with taking into account the related uncertainty.

When showing wave forms of the design basis tsunamis as time histories, the tsunamis in the coastal area away from the facility should be provided in order to minimize influence of reflected waves from the facility to time history wave forms, taking into consideration features of submarine topography in the sea area in front of the site. Regarding surveys in establishing the design basis tsunamis, survey methodologies should be selected so as to be suitable for survey objectives, and it is required to assure reliability and accuracy of survey results by considering application conditions, accuracy, etc. of the survey methodologies.

2. The “design basis tsunamis” in the above Item 1 should be established in accordance with the following policies:
 - (1) The following causes of tsunami should be taken into consideration, and it is required to select multiple causes that expect to have great influence on the site. Further, it is required to consider interpolate earthquakes and other types of earthquake, or a combination of earthquake and landslide or slope collapse, etc. on the basis of geological background of the site relating to causes of generating tsunamis, and relationships of these causes:
 - Inter-plate earthquakes.
 - Oceanic intraplate earthquakes.
 - Crustal earthquakes due to active faults in sea area.
 - Landslide on land and seabed, and slope collapse.
 - Volcanic phenomena (eruption, collapse of volcanic edifice or caldera subsidence, etc.)
 - (2) An appropriate scale of tsunami source should be assumed as estimated from plate geometry, slip deficit distribution, fault geometry, topography and geology, location of volcano, etc. In assuming the scale, it is required to consider similarity in mechanisms of generating tsunami and tectonic background, referring to domestic and overseas cases of large-scale tsunamis. In addition, it is required to refer to domestic and overseas cases of far-field tsunamis
 - (3) Concerning inter-plate earthquakes, it is required to consider earthquakes whose source areas are extending from the lower limit of seismogenic zone depth to trench axis.
 - (4) If changes in offshore seawater level that are caused by large-scale tsunamis occurring in other regions were recorded, it is required to investigate influence on the design basis tsunamis, if necessary, in consideration of similarity in mechanisms of generating tsunami and tectonic background, and effects of geography in the sea area where the above-mentioned changes were recorded,
 - (5) Run-up tsunamis due to the design basis tsunamis should exceed tsunami heights and flooded areas to be estimated from geological evidence such as tsunami deposits around the site, and historical records. If administrative organizations have assessed tsunamis near or around the site, their details should be reviewed, paying attention to differences in concept to determine tsunami sources, analytical conditions, etc., and then, it is required to reflect scientific and technological findings necessary from a viewpoint of making the assessment on the safe side, for establishing the design basis tsunamis.
 - (6) In order to have a sufficient margin in a tsunami-resistant design, it is required to adopt suitable methodologies, in considering uncertainty involved in process of generating the design basis tsunamis, by taking into account causes of uncertainty in determining tsunami sources that are thought to have large impacts on development of the design basis tsunamis (location, length, width, strike, tilt angle, slippage, slip angle, slip distribution, rupture initiation point, rupture

- propagation speed, etc. of fault), and degree of uncertainty and uncertainty due to differences in concept and interpretation relating to their uncertainty.
- (7) An extent of areas to be surveyed necessary to evaluate tsunamis should be set much wider than that to evaluate earthquake ground motions. It is required to conduct surveys by appropriately combining survey of past literature, geomorphological survey, geological survey, geophysical survey, etc. so as to take advantages of characteristics of these surveys, depending upon topographical and geological characteristics of the survey areas. Further, the following surveys should be carried out: survey on causes to generate tsunamis, survey necessary to develop tsunami source models, survey on tsunamis having the possibility of attacking surrounding of the site, and survey necessary to evaluate sand movement.
 - (8) Surveys and evaluations to establish the design basis tsunamis should be made on the basis of the latest scientific and technological findings. Past documents should be referred to in sufficient consideration of their sufficiency and accuracy, recognizing the ranges surveyed. If a view or approach different from that cited in the past documents is adopted, its basis or evidence should be clearly shown.
 - (9) It is required to grasp an exceedance probability of the design basis tsunamis established, by referring to exceedance probabilities of tsunamis corresponding to the design basis tsunamis.
3. In order to meet the requirement “should have no possibility of damaging safety functions” specified in Article 5, the facilities for design basis events should be designed against the design basis tsunamis in accordance with the following policies:
- (1) Regarding the site where Seismic class S facilities are installed (excluding facilities for protecting from tsunami, equipment for preventing flooding and tsunami monitoring equipment. The same exclusion is applied in Item (3) below.), it is required to protect the site from arrival or intrusion of run-up waves induced by the design basis tsunamis from the ground. Further, it is required to prevent their intrusion through intake and discharge channels, etc. The above requirements should be met in accordance with the following policies:
 - (a) It is required to install buildings containing Seismic class S equipment (excluding equipment for preventing flooding and tsunami monitoring equipment (the same exclusion is applied in Item (3) below.) and Seismic class S equipment (limiting to the equipment installed outdoors) on a place with such a high altitude that run-up waves induced by the design basis tsunamis are impossible to reach. If those are installed on a place that the run-up waves are possible to reach, it is required to install facilities for protecting from tsunami such as seawall, and equipment for preventing flooding.
 - (b) Concerning preventing arrival of run-up waves at the site, it is required to investigate the possibility of running up to the site including its wraparound, in consideration of topology and altitude of the site and its vicinity, presence of

ivers, etc., and wide-area upheaval and subsidence caused by an earthquake. If it may be possible to cause a change in topography or river course by deformations to be caused by an earthquake or scouring and deposition induced by repeatedly attacking tsunamis, it is required to investigate their impacts on routes of running up to the site.

- (c) It is required to investigate the possibility of intrusion of tsunami through channels such as intake and discharge channels, and then to identify paths (door, opening, penetration, etc.) with the possibility of intrusion. Intrusion of tsunami should be prevented by means of taking measures for preventing intrusion through these paths.
- (2) It is required to prevent damages of important safety functions by means of restricting a flooded area induced by leakage, referring to investigation on the possibility of leaking at intake and discharge facilities, underground structures, etc. This requirement should be met in accordance with the following policies:
- (a) It is required to investigate the possibility of water leakage at intake and discharge facilities, and underground structures, etc. in consideration of structural features, etc. of intake and discharge facilities, and then to assume a flooded area (hereinafter, referred to as “assumed flooded area”), supposing that the water leakage continues. At the boundary of the assumed flooded area, it is required to identify routes, which have the possibility of flooding, and flooded ports (door, opening, penetration, etc.). Based on these results, it is required to restrict the flooded area by taking flooding measures, which have the possibility of flooding.
 - (b) If there are Seismic class S facilities in the vicinity of the assumed flooded area, the area should be designed as watertight compartments. An amount of flooded water should be assessed as needed, and then it is required to confirm no possibility of affecting safety functions.
 - (c) If there is the possibility of long-term flooding in the assumed flooded area, it is required to provide drainage equipment.
- (3) It is required to isolate Seismic class S facilities including the facilities specified in (2) above from impacts, etc. induced by a tsunami by means of protecting them from flooding. In order to satisfy this requirement, it is required to clarify compartments and buildings containing Seismic class S equipment as priority areas for protecting from flooding. The flooded areas and amount of flooded water should be assumed on the safe side in consideration of overflow induced by the tsunami. It is required to identify routes, which have the possibility of flooding to the priority areas for protecting from flooding, and flooded ports (door, opening, penetration, etc.). Based on these results, it is required to restrict the flooded area by taking flooding measures, which have the possibility of flooding.
- (4) It should be prevented to damage important safety functions to be caused by reduction of seawater intake capability due to fluctuation in seawater level.

To meet the requirement, seawater pumps of the emergency seawater cooling systems should be designed to keep their functions against drawdown (or seawater level drop), and to maintain seawater necessary to cool. In addition, it is required to ensure water permeability at seawater intake ports and intake channels against movement and deposition of sand accompanied by water level fluctuation due to the design basis tsunamis, and drifts. Further, seawater pumps should be designed to keep their functions against sand inclusion through seawater intake ports.

- (5) Facilities for protecting from tsunami and equipment for preventing flooding should be designed to maintain functions to protect from tsunami and to prevent flooding against an input tsunami. The input tsunami is defined for individual facility by considering propagation properties of the tsunami, flooding routes, etc. in order to design the facility against the tsunami. Tsunami monitoring equipment should be designed to maintain functions to monitor the tsunami against the input tsunami. In order to meet the requirements, the facilities and equipment should be designed in accordance with the following policies:
 - (a) The above-mentioned facilities for protecting from tsunami include seawall, embankment structure and tide barrier. The equipment for preventing flooding mentioned above includes watertight doors, and flooding prevention equipment to be installed at openings and penetrations. Tsunami monitoring equipment mentioned above includes tide gages on the site, intake pit water level gages, and outdoor monitoring cameras that are capable of grasping situations of tsunami strikes. Other than the above-mentioned facilities and equipment, there are facilities and equipment for reducing impacts due to tsunami, such as breakwater, which is expected to reduce impacts on facilities for protecting from tsunami and equipment for preventing flooding that is caused by wave forces.
 - (b) Input tsunamis should be expressed as time history wave forms that are calculated at the installation location of individual facility and equipment, by conducting a numerical analysis for the area from the wave source of the design basis tsunami. The following should be taken into consideration in conducting the numerical analysis: site geometry, sea floor topography in the coastal area, angle of tsunami invasion to the site, existence of rivers, run-up and propagation effects of the land, artificial structures on propagation routes, etc. Further, it is required to adequately evaluate and consider a local natural frequency excitation of sea surface in a harbor, which is caused by a tsunami (author's commentary: this phenomenon is called one of secondary undulation).
 - (c) Facilities for protecting from tsunami should be designed to sufficiently maintain function for protecting from tsunami against the input tsunami, based on evaluation of resistance to erosion and scouring due to wave forces, and of stability against slipping and overturning, depending upon their structures, and in consideration of resistance during overflow.
 - (d) Equipment for preventing flooding should be designed to sufficiently maintain functions for preventing flooding against the input tsunami, based on evaluation resistance, etc. to a wave pressure, etc. during the flooding and after submerged

in the flood in the assumed flooded area, etc., and in consideration of resistance during overflow.

- (e) Tsunami monitoring equipment should be designed to sufficiently maintain tsunami monitoring functions against the input tsunami, on the basis of investigating installation of the equipment at locations where less affected due to tsunami effects (wave force, collision of drifts, etc.), and measures to prevent or mitigate their effects.
 - (f) If there is the possibility of failure, collapse and drifting of buildings & structures, installations, etc. inside the power station site and in its vicinity outside the facilities for protecting from tsunami, it is required to take measures to prevent drifting or to prevent impacts on the facilities for protecting from tsunami and equipment for preventing flooding, in order to prevent ripple effects on the facilities for protecting from tsunami such as seawall and equipment for preventing flooding.
 - (g) In designing as mentioned in (c), (d) and (f) above, etc., loads corresponding to function damage mode for each facility and equipment (flooding height, wave force, wave pressure, scouring force, buoyancy, etc.) should be determined with sufficient margins, starting from providing a margin to the input tsunami, in order to have a sufficient margin in tsunami-resistant design. It is required to take into account a combination of loads due to aftershock and those due to the input tsunami, as needed, on the basis of investigating the possibility of aftershock occurrence. Further, it is required to investigate effects of repeated tsunami attacks on the facilities protecting from tsunami and equipment for preventing flooding, on the basis of time history wave forms of the input tsunamis.
 - (h) It is required to design facilities and equipment for reducing effects of tsunami to keep functions to reduce the effects of tsunami for the input tsunami, and to meet the requirements (f) and (g) above, if it is required to take into consideration influences of these facilities and equipment in designing the facilities for protecting from tsunami and equipment for preventing flooding.
- (6) The following effects caused by earthquake and tsunami should be taken into consideration: uplift and subsidence of the site caused by earthquake, effects due to earthquake (mainshock and after shock), effects due to repeated tsunami attack, and secondary effects due to tsunami (scouring, sand movement, drift, etc.).
- (7) A design of the facilities for protecting from tsunami and equipment for preventing flooding and an evaluation of emergency seawater cooling systems should be made on the safe side, regarding seawater level fluctuations caused by the input tsunami in consideration of mean tide level. Further, tide level fluctuations produced by other causes should be appropriately considered. In addition, if an earthquake is expected to cause uplift or subsidence of the land, its evaluation should be made on the safe side, in consideration of an amount of crustal movement of the site, which is calculated using a seismic source model applied to the expected earthquake.

[2] Rules for ground, prevention of damage due to earthquake and that due to tsunami for facilities for coping with severe accidents, etc.**[Ground for facilities for coping with severe accidents, etc.]****Rules of the Ordinance*****Article 38******Subarticle 1***

The facilities for coping with severe accidents, etc. should be established on the ground to meet the following requirements in accordance with the facility classification as described below.

- (1) Facilities for coping with severe accidents, etc. (excluding specialized safety facilities) where among permanently installed equipment for preventing severe accidents (hereinafter, referred to as “permanent equipment for preventing severe accidents”), those substituting the functions that equipment for coping with design basis events belonging to seismically important facilities have (hereinafter, referred to as “seismically important and permanent equipment for preventing severe accidents”) are installed.

The ground with sufficient capacity for facilities for coping with severe accidents, etc. concerned against seismic forces due to the design basis earthquake ground motions].

- (2) Facilities for coping with severe accidents, etc. (excluding specialized safety facilities) where permanent equipment for preventing severe accidents are installed, excluding seismically important and permanent equipment for preventing severe accidents

The ground with sufficient support capacity for facilities for coping with severe accidents, etc. concerned against seismic forces that are calculated according to rules specified in Subarticle 2, Article 4.

- (3) Facilities where permanently installed equipment for mitigating severe accidents (hereinafter, referred to as “permanent equipment for mitigating severe accidents”) (excluding specialized safety facilities) are installed

The ground with sufficient support capacity for facilities for coping with severe accidents, etc. against seismic forces due to the design basis earthquake ground motions.

- (4) Specialized safety facilities

The ground with sufficient support capacity for specialized safety facilities concerned against seismic forces that are calculated according to rules specified in Subarticle 2, Article 4, and those due to the design basis earthquake ground motions.

Subarticle 2

Facilities for coping with severe accidents, etc. (excluding those specified in Item (2) in the above Subarticle. The same exclusion is applied in the next Subarticle and in Subarticle 2 in the next Article) should be established on the ground where there is no possibility of damaging functions necessary to cope with severe accidents, etc., even if deformed.

Subarticle 3

Facilities for coping with severe accidents, etc. should be established on the ground with no possibility of producing displacements.

Rules of the Interpretation

1. It is required to conform to the Separate Note No.1 of the current regulation for application of Article 38.
2. “Seismic forces that are calculated according to rules specified in Subarticle 2, Article 4” that is stipulated in Item (2) in Subarticle 1 above are equivalent to the seismic forces that are applied to the seismic class of equipment for coping with design basis accidents having the functions which the above permanent equipment for preventing severe accidents substitutes.
3. “Seismic forces that are calculated according to rules specified in Subarticle 2, Article 4” that is stipulated in Item (4) in Subarticle 1 above are equivalent to the seismic forces that are applied to Seismic class S facilities, as given in the Separate Note No.2, Item 1 of the current regulation.

[Prevention of Damages Due to Earthquakes]

Rules of the Ordinance

Article 39

Subarticle 1

Facilities for coping with severe accidents, etc. should meet the following requirements according to the facility classification as mentioned below.

- (1) Facilities for coping with severe accidents, etc. where seismically important and permanent equipment for preventing severe accidents is installed (excluding specialized safety facilities)

There should be no possibility of damaging functions necessary to cope with beyond design basis accidents with the possibility of escalating to severe accidents (or beyond design basis accidents that may lead to severe accidents, but not severe accidents) against seismic forces due to the design basis earthquake ground motions.

- (2) Facilities for coping with severe accidents, etc. where permanent equipment for preventing severe accidents is installed except seismically important and

permanent equipment for preventing severe accidents (excluding specialized safety facilities).

The facilities are required to withstand with a sufficient margin against seismic forces that are calculated according to the rules specified in Subarticle 2 in Article 4.

- (3) Facilities for coping with severe accidents, etc. where permanent equipment for mitigating severe accidents is installed (excluding specialized safety facilities)

There should be no possibility of damaging functions necessary to cope with severe accidents against seismic forces due to the design basis earthquake ground motions.

- (4) Specialized safety facilities

The facilities are required to withstand with a sufficient margin against seismic forces that are calculated according the rules specified in Subarticle 2 in Article 4, and to have no possibility of damaging functions necessary to cope with severe accidents, etc. against seismic forces due to the design basis earthquake ground motions.

Subarticle 2

Facilities for coping with severe accidents, etc. are required to have no possibility of damaging functions necessary to cope with severe accidents, etc. against slope collapse that may occur due to the earthquake specified in Subarticle 3 in Article 4.

Rules of the Interpretation

1. It is required to conform to the Separate Note No.2 of the current regulation for application of Article 39.
2. “Seismic forces that are calculated according to the rules specified in Subarticle 2 in Article 4” that is specified in Item (2) in Subarticle 1 above are equivalent to seismic forces that are applied to the seismic class of equipment for coping with design basis accidents having functions which the above equipment substitutes, as given in Subarticles 2 through 4 in Article 4 in the Separate Note No.2 (that is, Separate Note No.2, 4(2)–(4)) of the current regulation.
3. “Seismic forces that are calculated according the rules specified in Subarticle 2 in Article 4” that is specified in Item (4) in Subarticle 1 above are equivalent to seismic forces that are applied to the Seismic class S facilities as given in Item 1, Subarticle 2, Article 4 in the Separate Note No.2 of the current regulation.
4. In applying the requirement “to have no possibility of damaging functions necessary to cope with severe accidents, etc. against seismic forces due to the design basis earthquake ground motions” to the specialized safety facilities, as specified in Item (4) in Subarticle 1, allowable limits against the design basis earthquake ground motions to be stipulated in design standards are required to be identical to the allowable limits to be applied to facilities for design basis events. Further, it is required to enhance robustness against such an earthquake ground motion that exceeds the design basis earthquake ground motions to a certain extent, for instance, by taking different measures as their features from these for facilities for design basis events (so-called diversity as measures).

Such an example of the diversity is an application of a seismic isolation structure or vibration control structure to equipment belonging to the specialized safety facilities, in the case where a rigid structure is applied to equipment for coping with design basis events.

5. In applying the requirement stipulated in Item (4) in Subarticle 4 above, the related equipment, etc. such as indirect support structures necessary to keep functions of the specialized safety facilities is required not to lose their functions against seismic forces that are required to apply to the specialized safety facilities.

[Prevention of Damages Due to Tsunamis]

Rules of the Ordinance

Article 40

Facilities for coping with severe accidents, etc. should have no possibility of damaging functions to be required to cope with severe accidents, etc. against the design basis tsunamis.

Rules of the Interpretation

1. It is required to conform to the Separate Note No.3 of the current regulation for application of Article 40.
2. In applying the requirement “should have no possibility of damaging functions to be required to cope with severe accidents, etc. against the design basis tsunamis” to specialized safety facilities, as specified in Article 40, allowable limits against the design basis tsunamis to be stipulated in design standards are required to be identical to the allowable limits to be applied to facilities for design basis events. It is required to enhance robustness against such a tsunami that exceeds the design basis tsunamis to a certain extent, for instance, by taking different measures as their features from these for facilities for design basis events (so-called diversity as measures).

Such an example of the diversity is to contain the relevant facilities in a building with guaranteed watertightness or that is established on a hill.

Rules with regard to the seismic design and tsunami-resistant design that are given in the Ordinance and Interpretation are also provided in the following articles and their interpretations of the “Ordinance on Technical Standards for Commercial Nuclear Power Reactors and their Auxiliary Facilities”:

- Article 4 “Ground for facilities for design basis events,”
- Article 5 “Prevention of damage due to earthquake,”
- Article 6 “Prevention of damage due to tsunami,”
- Article 49 “Ground for facilities for coping with severe accidents, etc.,”
- Article 50 “Prevention of damage due to earthquake,”
- Article 51 “Prevention of damage due to tsunami,” and
- their Interpretations.

7.5 Practical Aspects of Seismic Design

7.5.1 *Methods for Obtaining Seismic Forces*

In the structural design expounded in Chaps. 5 and 6, pressure, thermal load, etc. were treated as static loads in many cases. Meanwhile, in seismic design, particularly in the case of Seismic class S facilities, it is stipulated that the seismic force (or the seismic load) shall not only be treated as static seismic force, but also be evaluated, by determining the design basis earthquake ground motion and the elastically dynamic design (or simplified as “elastic design”) earthquake ground motion, as dynamic seismic force generated thereby. Therefore, in order to understand the practical aspects of seismic design, it becomes important to understand how dynamic seismic force should be treated in addition to static seismic force. From such a viewpoint, the basic idea of the dynamic seismic response analysis will be described in this section, while expounding an important notion pertaining to static seismic force.

[1] About static seismic force

First, briefly touch the history of the Building Standards Act to be applied to general buildings.

Concepts of static horizontal seismic intensity for calculation of static seismic forces were proposed long ago, probably after the 1908 Messina Earthquake that happened in Italy. It should be noted here that the term “seismic intensity” when it is used in reference to earthquake is generally remindful of a seismic intensity meaning a strength of earthquake ground motion at a certain point or a seismic intensity scale. However, the seismic intensity that is used here is different from the above. The different definition of the seismic intensity that is used here is given later.

In Japan, the year after the 1923 Taisho Kanto Earthquake (the earthquake happened in the Taisho period in Kanto district in 1923), 1924, an allowable stress degree seismic design method (to restrict stress degrees (or unit stresses) within an allowable stress degree in order to design a structure within the elastic regime, according to linear elastic theory) was adopted in the Ordinance for Enforcement of the Urban Building Law, in which a seismic force was regulated for the first time in Japan. At that time, the horizontal seismic intensity was set as 0.1. It was raised to 0.2 in 1947, corresponding to the raise of allowable stress degree. In the Building Standards Act enacted in 1950, the horizontal seismic stress intensity was set to 0.2. Thereafter, reflecting lessons learned from the 1968 Tokachi Oki Earthquake (M 7.9), the 1978 Miyagi Prefecture Oki Earthquake (M 7.4), etc., the Act was drastically modified as shown below:• Introduction of the following concepts of primary design and secondary design

Primary design (so-called damage limit regulation); for medium-scale earthquakes, apply the conventional allowable stress degree design.

Secondary design (so-called safety limit regulation); for large-scale earthquakes, apply a horizontal load-carrying capacity design

- Introduction of the concept of horizontal load-carrying capacity
- Change of the static seismic force calculation method from the horizontal seismic intensity to the earthquake layer shearing force coefficient method.

As mentioned earlier, the Seismic Design Review Guide was originally issued in 1978. Incorporating the above-mentioned revisions, this Guide was modified in 1981. In this book, therefore, the 1981 edition is referred to as the Seismic Design Review Guide.

Later, the performance-based Building Standards Act was endorsed in 2000, with incorporating a concept of ultimate strength (or ultimate load-carrying capacity), on the basis of lessons learned from the 1995 Southern Hyogo Prefecture Earthquake. In calculating the ultimate strength, a design earthquake ground motion is assumed on the upper surface of engineering basement as the response spectrum for a dynamic seismic force.

Here, the engineering basement is defined as the ground with shear wave velocity $V_s \approx 300\text{--}700$ m/s (in some cases, 400 m/s), and is considered to have enough rigidity and strength to support buildings.

It should be noted further that magnitude of shaking of a propagating seismic wave varies depending upon the ground properties: the shaking is amplified in a soft ground. It is desirable in estimating an earthquake ground motion that the motion from a seismic source to a depth where it is not significantly affected by the ground is determined as a function of earthquake magnitude and distance from the seismic source, while for the portions where it varies depending on the ground, it is determined, taking into consideration the amplification properties. The ground surface at the depth where it is not affected by the ground is called the surface of the seismic basement. In many cases, the seismic basement is defined as the stratum with $V_s = 3$ km/s. However, since the seismic basement is located deep in the ground, there are not many observation records at the seismic basement and findings on underground structure in the range from the ground surface to the seismic basement. As a result, the engineering basement is used in lieu of the seismic basement in the Building Standards Act. The definitions of the seismic basement and engineering basement are provided in Sect. 7.3. Here, their somewhat different definitions are given.

Returning to the subject on static seismic force, a dynamic seismic force means, in plain words, a dynamic inertia force as an external force applied to a structure, etc. Although an earthquake is one of dynamic phenomena, the inertia force may be replaced with the equivalent static external force if the dynamic seismic force may be treated as the static seismic force. Conceptually speaking, if a building is modeled as a lumped mass, the following replacement may be assumed by replacing on the safe side: a dynamic external force equivalent to a dynamic inertia force $\Rightarrow$ a static external force $\Rightarrow$ a static seismic force.

It should be noted here that dynamic seismic forces are considered to be applied in two horizontal directions and in a vertical direction. A building is usually a multi-layered structure (i.e., has multiple floors), and its weight is dominated by the floors or ceilings. As shown in Fig. 7.16, the inertia force in the horizontal direction is assumed to apply to its central point. In other words, the inertia force due to the floor

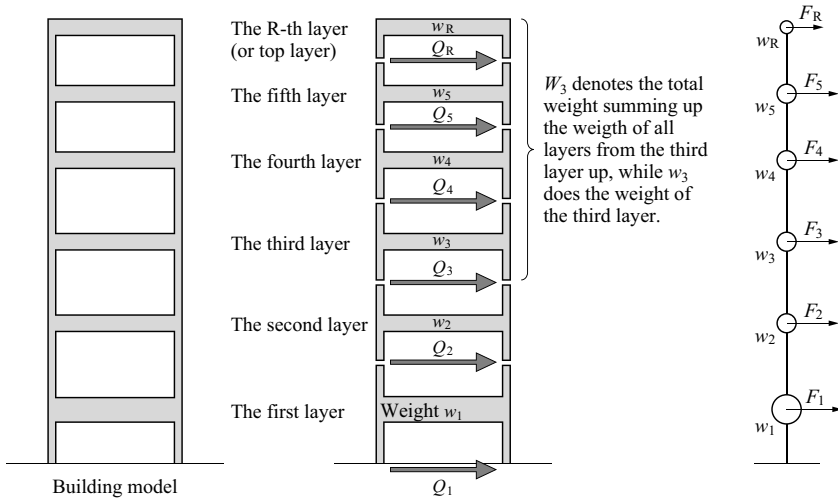
weight applies to the respective floor of the building, as the external force. Regarding fracture modes of the building, a shear fracture is important, while a bending fracture also is expected to occur. Consequently, an attention is paid to a shear force applied to each layer of the building. The shear force applied to each layer is not equal to the inertia force applied to the floor of each layer, but is represented by summing up the inertia forces applied to all layers from the layer concerned up. Two approaches are capable to calculate the shear force applied to each layer: one is the static seismic intensity approach and another is the earthquake (or seismic) layer (or story) shearing (or shear) force coefficient approach.

Here, the definition of the term “seismic intensity” is provided.

Let's consider a case where the base stratum on which a structure with mass M is situated shakes with a horizontal acceleration α . As discussed below, the structure is subsequently subjected to an inertial force expressed by the following formula, if approximated by one point lumped mass.

$$F = M\alpha \quad (7.5)$$

Since the weight W is expressed by $W = Mg$ (g : gravitational acceleration), this inertial force, and therefore, the seismic force, becomes



How to calculate the layer shear force

a) Earthquake (or seismic) layer (or story) shearing (or shear) force coefficient approach

$$\begin{aligned} Q_i &= C_i W_i \\ &= C_i \sum_{k=i}^n w_k \end{aligned}$$

b) Static seismic intensity approach

$$Q_i = \sum_{k=i}^n F_k$$

Horizontal (or lateral) force $F_i = Q_i - Q_{i+1}$

Fig. 7.16 Concept of layer shear force Q_i

$$F = \frac{W}{g} \alpha \quad (7.6)$$

This formula can be rewritten as [68]

$$F = W \left(\frac{\alpha}{g} \right) \quad (7.7)$$

Here, (α/g) is referred to as seismic intensity and expressed as K . Then, the seismic force can be obtained from the seismic intensity K using the following formula.

$$F = WK \quad (7.8)$$

Now, consider the building as shown in Fig. 7.16. The weight of the k th layer (floor) is expressed by w_k , and then a horizontal force generated by an earthquake (i.e., an inertia force), F_k , is derived as follows:

$$F_k = \left(\frac{\alpha_k}{g} \right) w_k = K_k w_k \quad (7.9)$$

The layer shearing force applied to the i th layer, Q_i , is expressed as follows:

$$Q_i = \sum_{k=i}^n F_k \quad (7.10)$$

where n denotes the top layer. This is the procedure for calculating the layer shearing force by means of applying the static seismic intensity approach (or the horizontal seismic intensity approach).

On the other hand, in another approach, the earthquake layer shearing force coefficient approach, the i th layer shearing force is derived from the total weight W_i that is summed up the weight of all layers from the i th layer up, as follows:

$$Q_i = C_i W_i, \quad W_i = \sum_{k=i}^n w_k \quad (7.11)$$

where C_i represents the earthquake layer shearing force coefficient, and is expressed by the following equation,

$$C_i = Z R_t A_i C_0 \quad (7.12)$$

where Z is the seismic zoning coefficient that is a decrease rate for each zone, depending on a magnitude of earthquake assumed, which is determined on the basis of past records of earthquakes, etc. in the zone concerned. For the Tokyo Metropolitan zone, $Z = 1.0$.

R_i is the vibration characteristic coefficient that is a reduction coefficient for taking into consideration the vibration characteristics of the ground and building.

A_i is the distribution coefficient of the earthquake layer shearing force coefficient, which, roughly speaking, represents the distribution of layer shearing force in the height direction of the building, because an amplitude and response acceleration are larger in the upper layers of the building than those in the lower. It may correspond to a distribution of the layer shearing force coefficient that means the layer shearing force divided by the total weight that sums up the weights of all layers from the layer concerned to the top layer as described earlier. For the lowest layer, $A_i = 1.0$, while the higher the layer concerned, the larger A_i .

C_0 is the standard layer shearing force coefficient, representing the ratio of an acceleration applied to the building to the gravitational acceleration. The Building Standards Act stipulates that 20% or 0.2 is assumed as moderate-scale earthquake ground motions in the primary design, while 100% as large-scale earthquake ground motions in the secondary design (referring to the requisite horizontal load-carrying capacity to be mentioned later). It is required to conduct the elastic regime design for buildings subjected to moderate-scale earthquake ground motions, in order to prevent any damage. Hence, $C_0 = 0.2$ is assumed in the allowable stress degree design as the primary design, which corresponds to approximately 80–100 Gal as acceleration of the input earthquake ground motions.

The current procedure of primary design method is first to calculate seismic forces acting on buildings by applying the earthquake layer shearing coefficient approach as the seismic intensity method, and next to conduct the elastic regime design on the basis of the allowable stress degree design approach. Further, a graded approach is adopted to nuclear power reactor facilities. Different factors (3.0, 1.5 and 1.0) are multiplied to C_0 depending on the seismic importance of the facilities. As described earlier, the graded approach was established in the seismic design of Unit No.1 of the Tokai nuclear power station, and has been adopted as remaining unchanged to the 1981 & 2006 Seismic Design Review Guides and even to the new regulatory standards that are issued by the NRA. Note here that not limited to the above-mentioned rules, many rules were established on the basis of engineering judgments in major engineering fields. Major rules have been evolved as a result of efforts of rationalization and sophistication (for instance, the concept of design factor as mentioned in Chap. 5), while other rules remain unchanged from the beginning, or the first legislation. In the latter rules, their backgrounds cannot be probably found without going back in history.

Great importance of understanding structural behavior up to fracture of buildings and structures, components and piping systems, etc. subjected to a large-scale earthquake has been recognized by surveying damaged buildings, etc. at the 1968 Tohoku Oki Earthquake, the 1978 Miyagi Oki Earthquake and so forth. In order to assure no collapse or overturning of buildings subjected to a large-scale earthquake, it is required to check and compare with ultimate strength or requisite horizontal load-carrying capacity, or other fracture-related strengths and ductility. This is called “secondary design.” A large-scale earthquake to be given in the secondary design is assumed to cause a response acceleration of 1.0 G, namely 100% of the gravitational

acceleration, as a result of an acceleration of input earthquake ground motion of 300–400 Gal acting on buildings, etc., and their response magnification of 3–4. It is required to judge whether or not at an event of large-scale earthquake, horizontal load-carrying capacity of a building, etc. concerned, Q_u , exceeds requisite horizontal load-carrying capacity, Q_{un} . The horizontal load-carrying capacity means horizontal resistance of the building, etc. in such a limiting or ultimate situation as to form an expected collapse mode, corresponding to R in Eq. (5.1) in Chap. 5. The requisite load-carrying capacity is a horizontal seismic force applied to the building, etc. at the large-scale earthquake, corresponding to L in Eq. (5.1). The requisite horizontal load-carrying capacity can be obtained by applying the following expression:

$$Q_{un} = F_{es} \times D_s \times Q_{ud} \quad (7.13)$$

where F_{es} is the shape coefficient, representing shape characteristics of each layer (floor), and is expressed as a product of two incremental (or additional) coefficients, one depending on eccentricity, F_e , the other depending on rigidity, F_s . D_s is the structural characteristics coefficient, representing a reduction factor of toughness of a structure, namely, deformation capability. D_s is formulated as a ratio of a maximum response layer shearing force applied to a tough structure under evaluation to a shearing force applied to the structure that responds elastically. Q_{ud} is the layer shearing force to be calculated by applying Eqs. (7.11) and (7.12), assuming the standard layer shearing force coefficient C_0 equal to 1.0 or greater.

[2] Seismic Response Analysis for Evaluating Dynamic Seismic Forces: Basic Concept

An earthquake shakes the ground, the shaking of the ground produces acceleration of buildings and structures on the ground. If the structure is observed from another viewpoint, i.e., from the ground or the base stratum constituting a base for the structure, it appears as if the structure is shaking with acceleration. The acceleration produced by shaking the ground, and then the inertia force generated by this acceleration is directly applied to the structure as an external force. As shown in Fig. 7.17, among the methods of analyzing the seismic response of the structures are the method of analyzing the response of structures on the premise that the ground or base stratum shakes as it actually does, and the method wherein it is assumed that the ground or base stratum is fixed and the structures experience acceleration as an external force in the reverse direction to that of the ground.

Next, let's consider the analytical model for conducting response analysis. In plants like nuclear reactor facilities, the layout, configuration, etc. of components are very complicated, making it unrealistic to model them accurately. Therefore, the modeling that enables to express necessary responses with adequate accuracy needs to be contrived. Such a model is referred to as a mechanical model.

In the seismic response analysis, the basis of this modeling is, as illustrated in Fig. 7.18b, the model represented by a so-called beam (or spring) and lumped mass system, wherein the mass, stiffness, etc. of a structure are concentrated and configured as beams (or springs) and lumped masses. An actual structure is configured with

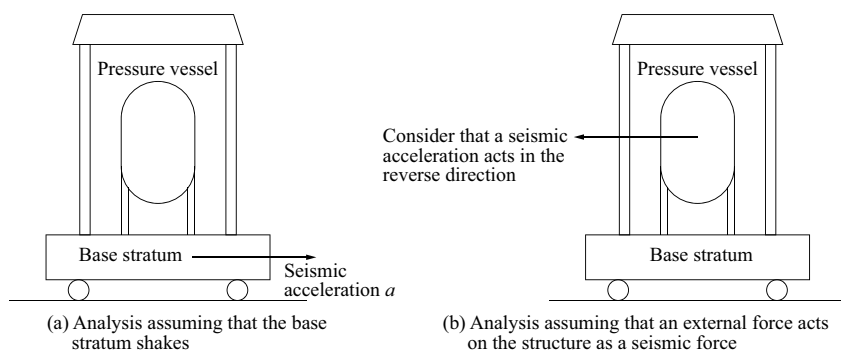


Fig. 7.17 Concept of seismic response analysis for structures

plural lumped masses and beams (springs), rather than a single-lumped-mass system illustrated in this figure. The locations of these lumped masses (i.e., the beam length in this figure) are often the locations where the response data of buildings, components, etc. are required in the designing stage. This model is also called a skewered dumpling model from its shape. The response analysis is conducted by inputting an earthquake ground motion to the foundation of the structure (in Figs. 7.17 and 7.18, the base stratum) using such a model.

However, when the natural period of a structure like a nuclear reactor building is short, the effect of damping due to dissipation in the ground becomes significant, and therefore it is necessary to take into consideration the interaction between the building and the ground. When it is necessary to analyze the interaction between the base of such a building and the supporting ground, a model called the sway-rocking spring model (S-R model, for short) is sometimes used. A response displacement of the building and structure is affected by a rigidity of the ground through the interaction, and the structure base additionally produces translational and rotational movements

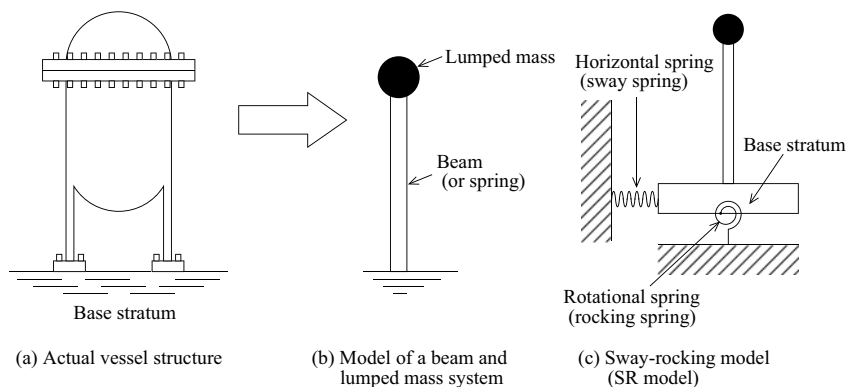


Fig. 7.18 Mechanical models for seismic response analysis

(sway and rocking, respectively) due to the interaction. As shown in Fig. 7.18, the sway spring, causing the sway movement, and the rocking spring, causing the rocking movement, are set between the structure and the ground, and are connected to the base of the structure. The other methods for modeling such interactions include a method wherein the ground is modeled using finite elements and structures like buildings are modeled as beam (or spring) and lumped mass systems, and a method wherein both of the ground and buildings/structures are modeled using finite elements and are coupled.

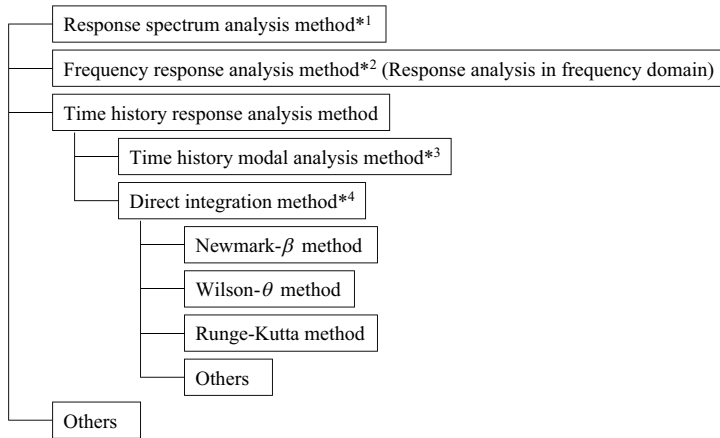
Using such a mechanical model, the responses of the structure to the design basis earthquake ground motion S_s and the elastically dynamic design earthquake ground motion S_d are obtained by dynamic analysis. Here, as shown in Fig. 7.19, the methods of dynamic response analysis can be largely classified into two categories, namely the time-history response analysis in which the resulting responses are obtained as time history, and the response spectral analysis that focuses on maximum responses, since obtaining maximum responses is sufficient for design in many cases. As a result of this response analysis, the distribution of response acceleration, displacement, shear force, bending moment, etc. generated in each portion of the structure are obtained. Stress analysis is conducted on the basis of these results, and seismic performance is evaluated by comparing the obtained stresses with the allowable limits.

7.5.2 Load Combinations and Allowable Limits in Component Design

Since it is difficult to predict when an earthquake occurs, the possibility that large-scale (or strong) earthquakes may occur, for example, during the occurrence of an accident, cannot be denied. For this reason, it is necessary to determine, on the basis of probability, what are the events whose loads should be combined with the design basis earthquake ground motion S_s or the elastically dynamic design earthquake ground motion S_d . As stated so far, this is specified by “load combinations.” The stress evaluation of buildings, equipment, etc. shall be conducted for the assumed combinations in order to prevent possible occurrence of accidents due to earthquakes and ensure that the general public would not be exposed to undue radiation risks.

Meanwhile, among buildings and structures or components and piping systems, there are cases where it may guarantee that the seismic safety mentioned above can be ensured by evaluating other parameters than material strength-related parameters such as stress, deformation and strain. An example is the case of pumps that need to continue rotating even when an earthquake occurs. In order to ensure such dynamic functions when it is not easy to provide analytical proof, it may become necessary to conduct confirmation testing for dynamic functions, as a way to confirm that the dynamic functions would not be lost even at an event of earthquake.

There is specificity in seismic force in contrast to the pressure and thermal loads in the nuclear reactor equipment. For this reason, when earthquake ground motions are



[Notes]

*1: (1) First, from a dynamic response analysis of simple oscillators (or single degree of freedom systems) of varying natural frequency, obtain their maximum (or peak) seismic responses (acceleration, velocity, displacement, etc.) as a function of natural frequency (or structural period) for a given time history and level of damping (in seismic analysis, the time history is an input earthquake ground motion),

(2) Then, generate the (seismic design) response spectrum as a plot of the maximum responses vs. natural frequency or structural period,

(3) Evaluate the natural frequency of a building or structure under analysis,

(4) Estimate its maximum response by reading the value from the response spectrum generated for the appropriate frequency, and

(5) Calculate seismic forces acting on the building.

It is not so practical to generate the response spectrum for each site on the basis of records of earthquake ground motions due to earthquakes that occurred at and around the site, in accordance with the above-mentioned procedure. It may be recommended that use the so-called “standard response spectrum.” The response spectrum for each site can be generated from the standard response spectrum by taking account of damping coefficient of the building or structure under analysis, characteristics of the site area, etc. For a complex building or structure, combination of the above results for many modes (computed through modal analysis) may be required. If a multi-degree of freedom (MDOF) system such as complex building is subjected to an earthquake, it undergoes deformation in number of possible ways. These deformed shapes are called modes of vibration or mode shapes. The modes are identified by the modal analysis. The frequencies and mode participation factors of the building as well as the mode shapes are obtained from the modal analysis. Total unique modes for each MDOF are equal to the possible degree of freedom of system. The maximum response of the complex building can be approximately estimated by combining the maximum response for each mode that is derived from the seismic design response spectrum for the single degree of freedom systems. The methods for combining the maximum for each mode for obtaining the maximum response of the MDOF system, so-called modal combination rules, include the following:

- i) Absolute sum (ABSSUM) method,
- ii) Square root of sum of squares (SRSS) method

The maximum response $r_{\max}$ is obtained by the following expression

$$r_{\max} = \sqrt{\sum_{i=1}^n s_i^2}$$

where s_i is the maximum response in the i -th mode.

- iii) Complete quadratic combination (CQC) method

The seismic forces are obtained as static seismic forces by means of seismic intensities that are determined from the response spectrum, or as dynamic seismic forces through a time history response analysis using ground motions that are calculated from the response spectrum.

Fig. 7.19 Typical methods for solving equations of motion in dynamic analysis

combined with other events (events resulting from internal factors or events generated dependently on earthquakes) and stress evaluation is conducted, it is necessary to take the specificity of seismic force into consideration appropriately.

(1) Regarding the combination of events, accordingly the combination of loads:

A strong earthquake occurs quite rarely, and therefore in terms of preventing the occurrence of accidents due to its occurrence, there is no rational basis in requiring that the component conditions similar to those required for the operating condition I or II should be maintained. However, there are components such as the reactor containment vessel that are required to maintain safety, for events pertaining to operating condition IV, as a design condition or at the level of the service condition A. Accordingly, when load combinations are examined, it is necessary to appropriately consider not only the events to be combined and the occurrence probability of a strong earthquake, but also in what conditions the components shall be maintained.

(2) Regarding stress evaluation:

Earthquakes are dynamic phenomena. As shown in Fig. 7.1, although earthquakes generally subside within about 1 min, it appears that the wave number of the seismic wave that occurs during this period amounts to 100 to 200. Meanwhile, the possibility that the pressure, temperature, etc. change significantly during this period is small. In cases where it is necessary to evaluate the failure caused by cyclic loads like seismic forces, the “design by formula” approach has difficulty in handling them, and therefore it is necessary to apply the “design by analysis” approach. In this case, it is necessary to address the issue as to how the specificity pertaining to the temporal change in seismic forces should be reflected in the design method and stress limits. The following description expounds how such issues are considered in the case of components and piping systems.

[1] Load combinations

As described in Sect. 7.3.5, the requirements that Seismic class S components and piping systems should fulfill are stipulated in “7. Load combinations and allowable limits” of the 2006 Seismic Design Review Guide, and as the new regulatory standards issued by the NRA, Articles 3 and 6 regarding facilities for design basis events, the Separate Note No.2, the “Interpretation of the Ordinance on Standards for Location, Structures and Equipment” (the full name of the Ordinance is the “Ordinance on Standards for Location, Structures and Equipment of Commercial Nuclear Power Reactors and their Auxiliary Facilities”). The basis for achieving this is that, regarding components of prevention systems, such as a reactor pressure vessel, “the allowable stress shall be determined in such a way that it would, in principle, not exceed the yield strength of the material regarding the sum of the primary stresses resulting from the combination of the normal load and the seismic force due to the elastically dynamic design earthquake ground motion S_d , as mentioned in the “2006 Seismic Design Review Guide.” In addition, even if the sum of the secondary and

the primary stresses exceeds the yield strength, it shall be confirmed that the deformation in that case would not damage the functions of the facility.” It appears that this has not been changed from the former guide [70]. This means that the allowable limits applied for the cases where an earthquake and other events are combined are different from the allowable limits used in usual structural design for the category relevant to the events to be combined with an earthquake. Starting from this basis, the occurrence probability of the events to be combined with an earthquake and the occurrence probability of the earthquake are taken into consideration, and subsequently considerations are given as to what the allowable limits would be against the stresses resulting from the combinations.

Meanwhile, the stress limits primarily set in structural design for each service condition are prescribed according to the states that should be maintained in the service condition. Accordingly, if the states that should also be maintained during an earthquake can be correlated with the case of structural design, in order to avoid unnecessary confusion, it is desirable that the correlation between the service conditions and stress limits, pursuant to those in structural design, can be applied to the allowable limits or stress limits against the combinations of the seismic force and other loads on the basis of the states that should be maintained against those combinations. However, it is necessary to consider appropriately the specificity pertaining to the seismic force. Since the requirements in the “Seismic Design Review Guide” including the basis described above indicate that the notions pursuant to those in structural design are applicable, the concept of the “allowable stress states,” which are based on the service conditions (or operating conditions) and also reflect the specificity of the seismic force, had been contrived in the course of examining the “1981 Seismic Design Review Guide.”

Accordingly, taking into consideration the occurrence probability of internally initiating events (those resulting from internal factors) (or the occurrence probability of events generated dependently on earthquakes) and the occurrence probability of an earthquake, if the state that should be maintained when an earthquake occurs during the internally initiating event corresponds to a service condition category X (X is A, B, C, or D), then let's refer to the allowable stress state as X_s here. In X_s , the stress limit pursuant to the service condition X shall be applied as a basis, and in addition to this, the specificity pertaining to the seismic force shall be taken into consideration. (Note that the term “allowable stress state” has not been widely acknowledged and it is still temporary. Incidentally, in the “1981 Seismic Design Review Guide,” this was referred to as III_{AS} when associated with the operating condition III.) The allowable stress states are indicated in Table 7.8, for cases where the elastically dynamic design earthquake ground motion S_d or the design basis earthquake ground motion S_s is generated when the category of the internally initiating event corresponds to the service condition A, B, C or D in structural design. The table shows the cases for a reactor pressure vessel and a reactor containment vessel. The readers are recommended to consider, as an exercise, the cases for other components.

Table 7.8 Example of classification for allowable stress states

(a) Reactor pressure vessel						(b) Reactor containment vessel					
Earthquake ground motion	Service conditions for internally initiating events					Earthquake ground motion	Service conditions for internally initiating events				
	A	B	C	D			A ^{***}	B	C	D(S) ^{**}	
				D(L) [*]	D(S) ^{**}						
S_d	C _s	C _s	C _s	D _s	—	S_d	C _s	C _s	C _s	—	—
S_s	D _s	D _s	D _s	—	—	S_s	D _s	D _s	D _s	—	—

* D(L) is the case where the event persists for a long period of time.

** D(S) is the case where the event lasts for a short period of time.

*** This includes LOCA, etc.

[2] Design Methods and Allowable Limits (Stress Limits)

The design methods and stress limits applied to main components, etc. when the seismic force and other loads are combined are indicated in Table 7.9.

In this table, the reactor internals, embedded plates, pressure part tension bolts, etc. are omitted.

Table 7.9 was prepared based on the following ideas.

- (1) For seismic class 1 vessels, piping, a part of valves and core support structures of facilities for design basis events to which the “design by analysis” approach is applied in structural design, “design by analysis” shall be applied. However, the specificity of the seismic force shall be taken into consideration. More specifically, in order to prevent failure, such as ductile fracture that may be caused even by single load application, the stress obtained by adding the seismic force and other loads shall be evaluated for the primary stress limit, etc. Meanwhile, with regard to the primary plus secondary stress limits for preventing ratchet and fatigue failure that can be generated solely by cyclic loads as well as fatigue evaluation for this case, only the stresses due to the design basis earthquake ground motion S_s and the elastically dynamic design earthquake ground motion S_d shall be evaluated. Here, in the fatigue evaluation, the stresses due to loads other than seismic force, such as pressure, shall be treated in terms of mean stresses.
- (2) For components, etc. to which “design by formula” is applied in structural design, “design by analysis” shall be applied to the combinations with earthquakes. For example, when fatigue evaluation is carried out, only the stresses due to the design basis earthquake ground motion S_s and the elastically dynamic design earthquake ground motion S_d shall be evaluated, and the cumulative fatigue usage factor $U \leq 1.0$ due to these stresses shall be derived. The details of stress limits depend on components and seismic classes.

Table 7.9 Design methods and stress limits in the seismic design of components, etc. of facilities for design basis events

Classification of importance in the seismic design	Classification of components, etc. (Methods of structural design are shown in the lower part of the column of each class.)						
	Class 1					Class MC	
	Vessels	Piping	Pumps**	Valves**	Supports	Vessels	Supports
	“Design by analysis”	“Design by analysis”	“Design by formula”	“Design by formula” (rated pressure-temperature reference valve method), and simplified “design by analysis” in part	“Design by formula” incorporating the Architectural Institute of Japan’s “Design Standard for Steel Structures.” However, fatigue evaluation is unnecessary.	Basically, “Design by formula”	Pursuant to the prescription for Class 1 supports
S	“Design by analysis” considering the specificity of earthquake ground motions*	“Design by analysis” considering the specificity of earthquake ground motions*	For the combination with seismic force, the prescription for Class 1 vessels is applied.	Basically, evaluation is unnecessary.***	“Design by analysis” of Class 1 valves in structural design, considering the specificity of earthquake ground motions*. However, fatigue evaluation is unnecessary.	“Design by analysis” for the combination of seismic force with other loads, considering the specificity of earthquake ground motions. However, the allowances are different.†	(Not stipulated yet)
B & C	(NA)	(NA)	(NA)	(NA)	(NA)	(NA)	(NA)

(continued)

Table 7.9 (continued)

Classification of importance in the seismic design	Classification of components, etc. (Methods of structural design are shown in the lower part of the column of each class.)					
	Core support structures	Vessels	Piping	Pumps	Valves	Supports
	"Design by analysis"	"Design by formula"	"Design by formula"	"Design by formula"	"Design by formula" (rated pressure-temperature reference valve method)	Pursuant to the prescription for Class 1 supports against the primary stress in the service condition A and B
S	"Design by analysis" of core support structures in structural design, considering the specificity of earthquake ground motions (But, evaluation is unnecessary for primary + secondary stress limits and fatigue evaluation.)	"Design by analysis" for the combination of seismic force with other loads, considering the specificity of earthquake ground motions. However, the allowable stresses are different. ^{††}	Pursuant to the prescription for Class 2 vessels for the combination with seismic force.	Pursuant to the prescription for Class 2 vessels for the combination with seismic force.	Basically, evaluation is unnecessary.	Based on the prescription for Class 1 supports for the combination with seismic force.
B & C	(NA)	Simplified "design by analysis" for the combination of seismic force with other loads. However, the allowable stresses are different. ^{†††}	Basically pursuant to the prescription for Class 2 vessels for the combination of seismic force with other loads, and in addition, the shakedown evaluation (primary stress + secondary stress $\leq 2S_y$) shall be conducted.	Based on the prescription for Class 2 vessels for the combination with seismic force.		This shall be based on the prescription for Class 1 supports in seismic Class S for the combination with seismic force.

(continued)

Table 7.9 (continued)

Classification of importance in the seismic design	Classification of components, etc. (Methods of structural design are shown in the lower part of the column of each class.)				
	Class 3				Class 4 piping
	Vessels	Piping	Valves	Supports	
	"Design by formula"	"Design by formula"	Pursuant to the prescription for Class 2 valves	Pursuant to the prescription for Class 2 supports	Wall thickness, design of weld zones, etc are prescribed.
S	Based on the prescription for Class 2 vessels for the combination with seismic force.	Based on the prescription for Class 2 piping for the combination with seismic force.	Pursuant to Class 2 valves	This may be pursuant to Class 1 supports	Span lengths of supports shall be maintained below the buckling limit length against acceleration and relative displacement during earthquake.
B & C	Based on the prescription for Class 2 vessels for the combination with seismic force.	Pursuant to the prescription for Class 2 vessels for the combination with seismic force.	Pursuant to Class 2 vessels	This may be pursuant to Class 1 supports	Based on the prescription for Class 4 piping in seismic Class S

* "Design by analysis" shall be applied for the combination of seismic force and other loads. However, regarding the primary plus secondary stress limits and fatigue evaluation for preventing damage caused only by alternating loads, it is only necessary to evaluate the stresses due to the design basis earthquake ground motion S_e and the elastically dynamic design earthquake ground motion S_h , and it is not necessary to make evaluation for the sum of these stresses and stresses caused by other loads such as pressure, thermal load, etc.

** Regarding the functions of dynamic components such as pumps and valves for which operation functions are required during or after an earthquake, it shall be confirmed, through testing or suitable analysis, that these functions can be maintained and that the pressure resistance indicated in the above table can be ensured. Furthermore, it shall be confirmed that Class 1 pumps would not cause pump seizures. However, as for the Class 1 pumps in BWR and PWR, evaluation regarding the maintenance of operation functions is not required because there is no necessity.

*** It is not necessary to dare to evaluate the valve boxes for pressure resistance functions against the reaction force added from the connected piping, etc. during an earthquake. However, the evaluation of integrity during an earthquake shall be conducted for the valves connected to pipes with an outer diameter less than 115 mm.

† For cases of combination where the stress limits for the service condition C are applied, with the allowable stress S , the primary general membrane stress, for example, becomes $P_m \leq 1.2S$ according to structural design, but it shall be $P_m \leq \min(S_y, 0.6S_u)$. However, it shall be $1.2S$ for austenitic stainless steels and high nickel alloys. Regarding the stress limits for the service condition D, among the stress limit prescribed by the "design by analysis" method, the corresponding term pertaining to cases in which stress is generated by jet force and mechanical loads (applied in mechanical design for cases where significant stress is generated by specific loads within Class MC vessels) shall be used for the term on the right-hand side (related to the allowable stress) of the stress limit. More specifically, the intensities of the primary general membrane stress shall be as follows.

(i) $0.6S_u$ for structurally continuous portions. However, it shall be $\min(0.6S_u, 2S)$ for austenitic stainless steels and high nickel alloys.

(ii) $\min(S_y, 0.6S_u)$ for structurally discontinuous portions. However, it shall be $1.2S$ for austenitic stainless steels and high nickel alloys.

†† (a) For the combination of loads where the stress limit for the service condition C in structural design is applied:

(1) The primary general membrane stress $\sigma_m \leq \min(S_y, 0.6S_u)$. However, $\sigma_m \leq 1.2S$ for austenitic stainless steels and high nickel alloys.

(2) 1.5 times the above value for the primary membrane stress plus primary flexural stress.

(b) For combinations where the stress limit for the service condition D is applied:

(1) The primary general membrane stress $\sigma_m \leq 0.6S_u$.

(2) The primary membrane stress plus the primary flexural stress $\sigma_m + \sigma_b \leq 1.5 \times 0.6S_u$.

(c) Fatigue analysis shall be conducted for the cyclic peak stress consisting only of the stress due to the design basis earthquake ground motion S_e or the elastically dynamic design earthquake ground motion S_h , and the fatigue usage factor shall be $U \leq 1.0$. However, the shakedown evaluation is unnecessary.

††† (1) The primary general membrane stress $\sigma_m \leq \min(S_y, 0.6S_u)$. However, $\sigma_m \leq 1.2S$ for austenitic stainless steels and high nickel alloys.

(2) The primary stress $\sigma_p \leq S_y$. However, $\sigma_p \leq 1.2S$ for austenitic stainless steels and high nickel alloys.

7.5.3 Seismic Design and Analysis

After performing the sizing for maximum allowable working pressure (i.e., design pressure), etc. as component design, seismic supporting structures for enhancing seismic resistance shall be designed. Furthermore, after evaluating the thermal expansion stress due to the seismic support structures, etc., the analysis of earthquake strength (and also the analysis of seismic response by dynamic analysis, for Seismic class S components of facilities for design basis events.) shall be conducted for the seismic forces that have been obtained by inputting the design basis earthquake ground motions, etc. described in Sect. 7.3.3 in consideration of the combination of loads such as operating loads. Consequently, seismic safety shall be confirmed through comparison with the allowable limits described in Sects. 7.3.5 or 7.5.2. If by any chance the results did not fall below the allowable limits, this process shall be repeated by changing the seismic support structures. The flow of this seismic analysis is illustrated for facilities for design basis events in Fig. 7.2.

As a representative example of analytical model used for seismic response analysis, Fig. 7.20 shows the case of a steam generator. Pressure vessels, such as a steam generator, are not only supported by supports, but they are also connected with piping and therefore affected by other components. Accordingly, it is not the independent evaluation of a steam generator that is necessary, but the analysis of a coupled system including the supports and connected components need to be conducted. Generally, in analytical models, the stiffness of each portion is computed using a bending-shear beam model and the weight is modeled as a lumped mass system, constituting a multiple-lumped-mass system. The locations of lumped masses in the model are determined by considering the locations for attaching pipe nozzles, locations in structures with discontinuities changing in wall thickness, main points for stress evaluation, etc.

Exercises

1. Consider an inverted pendulum shown in the figure, as a vibration model of a structure that experiences an earthquake ground motion. Here, a lumped mass (mass = M) and a spring (mass = 0) supporting this shall represent the mass and stiffness of the real structure, respectively. In addition, let the parameters be as follows: the damping coefficient of the system C , the spring constant K , and the earthquake ground motion (displacement) on the Earth's surface x_e .
 - (1) Derive a differential equation for an unknown value x , which represents the relative displacement of a lumped mass with respect to the earth's surface.
 - (2) Explain the analysis method for the time-history response of the above-mentioned differential equation.

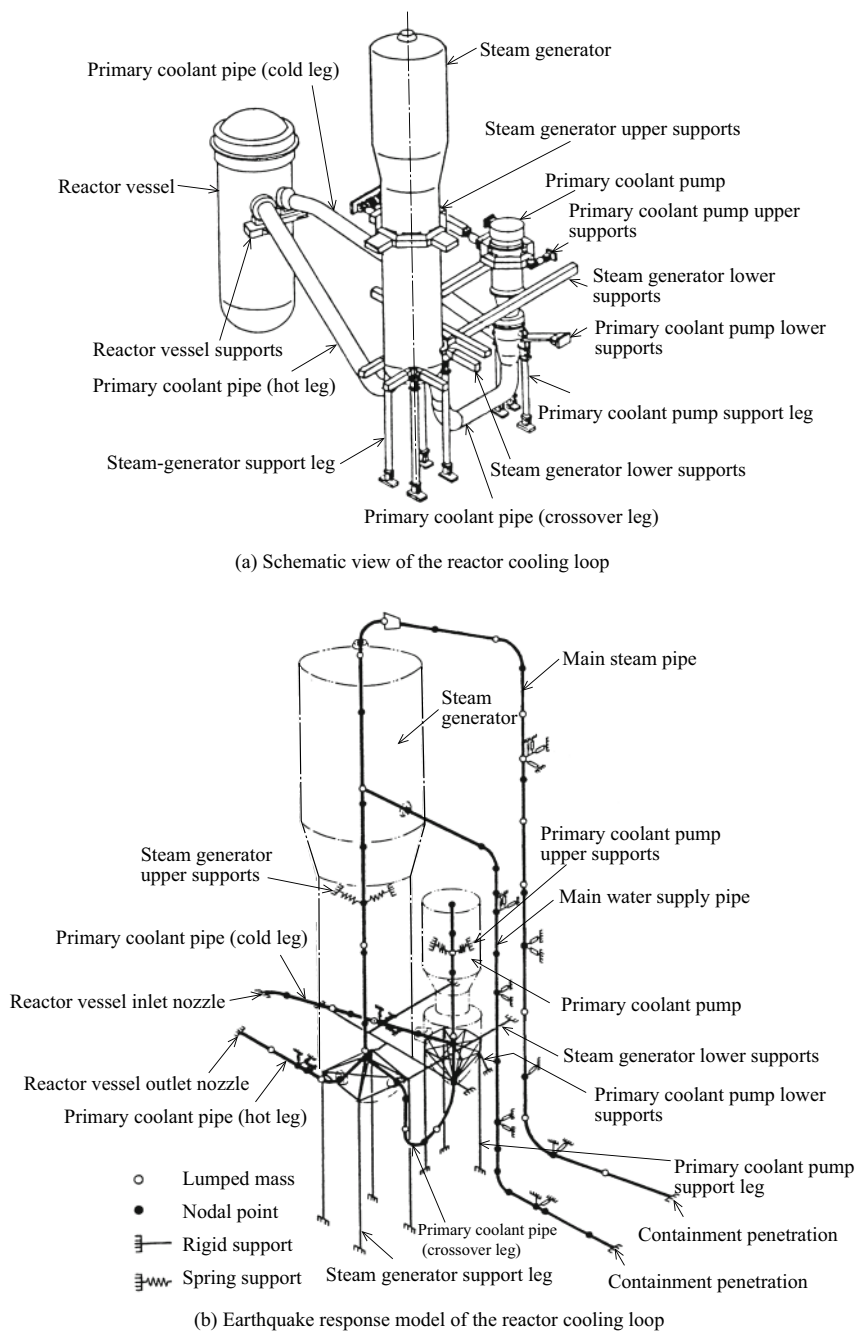
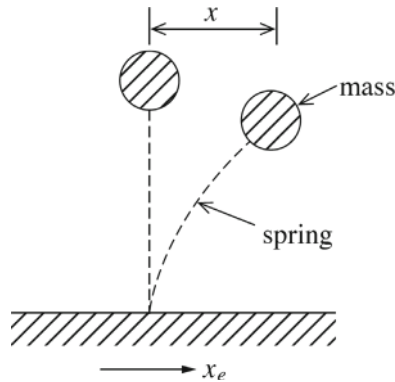


Fig. 7.20 Modeling of PWR reactor cooling loop [70]



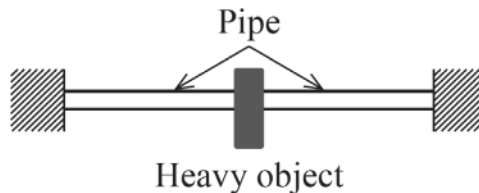
2. Answer the following questions regarding the Regulatory Guide for Reviewing Seismic Design of Nuclear Power Reactor Facilities.
 - (1) Explain the design basis earthquake ground motion S_s and the elastically dynamic design earthquake ground motion S_d within about 400 characters in total.
 - (2) Explain the dynamic and static methods for seismic computation within about 400 characters in total.
 - (3) Explain the classification of importance in the seismic design within about 400 characters.
3. Various components of nuclear power reactor facilities are usually installed on the floor surface of each story. A kind of response spectrum called a floor response spectrum is used for the seismic design of such components. Explain the concept of the method for developing a floor response spectrum, using a schematic diagram.
4. Regarding the method for obtaining a “site specific earthquake ground motion whose source to be identified” in order to determine the design basis earthquake ground motion S_s , explain the items that require examination, earthquakes that needs to be investigated, and the method for determining the earthquake ground motion.
5. Explain the following terms briefly.
 - (1) Active fault
 - (2) Residual risks
 - (3) Seismic class S facilities
 - (4) Combination of loads
 - (5) Free surface of the base stratum.
 - (6) Floor response spectrum
 - (7) Earthquake epiphenomenalism (or accompanying phenomena)
 - (8) Acceleration response spectrum
 - (9) Seismic source fault (including a difference from an active fault)

(10) Engineering basement and seismic basement

6. Explain approach and features of the fatigue evaluation method for evaluating stresses developed in components and piping systems in seismic design in Japan.
7. As shown in the figure below, consider such a piping system that a heavy object is fixed at both (left and right) ends, and an earthquake induces its repeated vertical vibration.

Answer the following questions regarding the natural frequency of the piping system. Its behavior can be expressed according to a linear elastic theory, and the natural frequency is proportional to the square root of its tensile rigidity divided by the mass of the heavy object.

- (1) If the mass of the heavy object is quadrupled, how does the natural frequency change?
- (2) If a material that the Young's modulus is twice as much is adopted for the piping, how does the natural frequency change?
- (3) If the diameter is twice as much while keeping the ratio of the wall thickness of the piping with the diameter, how does the natural frequency change?
- (4) If the length of the piping is twice as much, how does the natural frequency change?
- (5) If the outer diameter, inner diameter, total length, Young's modulus of the piping material, and mass of the heavy object are assumed 500 mm, 400 mm, 200 mm, 200,000 MPa, 10 tons, respectively, how much is the natural frequency? Select the closest value of the following:
(a) 0.5 Hz (b) 5 Hz, (c) 50 Hz, (d) 500 Hz.



8. Regarding the requirement for preventing damage due to an earthquake in the “Ordinance on Standards for Location, Structures, and Equipment of Commercial Nuclear Power Reactors and their Auxiliary Facilities,” facilities for design basis events are classified into 3 classes (S, B and C) in accordance with seismic importance. Describe the definitions of each classification class, and cite five specific examples of facilities belonging to Seismic classes S and B, respectively, for nuclear power reactors.

[Answer of example: illustrative only] Seismic class S: reactor containment vessel.

9. Describe concepts of establishing the following two levels of earthquake ground motions for a seismic design of nuclear power reactor facilities in Japan, focusing

on their historical changes from the Tsuruga Nuclear Power Plant Unit No. 1 and Mihama Nuclear Power Plant No.1 of the Kansai Electric Power Co., Inc. to date. In particular, about the relationship of both ground motions, geological surveys necessary to establish them, and reasons for changes.

- Earthquake ground motions required to confirm ensuring seismic safety of the most important facilities (examples of safety functions: for facilities for design basis events, controlling the power including emergency shutdown, cooling the fuels (or the core), and confining the radioactive materials (fission products), and further relating these three functions, tsunami resistance, in order to prevent causing such an event radiation exposure dose outside the facility exceeds 5 mSv)
 - Earthquake ground motions for elastically dynamic design (an objective is to size facilities and structures)
10. Investigate and summarize features and superiority/inferiority of the response spectrum method (using the so-called “Taisen’s spectrum” (“Taisen” is the abbreviation of the specialized subcommittee on seismic design) and the method using a fault model, respectively. Further, investigate how the “Taisen spectrum” is improved from the “Osaki spectrum,” including seismic source faults and control points for generating the spectrum.

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Chapter 8

Manufacture



This chapter summarizes the processes to manufacture nuclear components that have been designed on the basis of structural design techniques described in the previous chapters. The steps of manufacture will be described first, and according to the steps, selection of materials, manufacturing technologies, heat treatment, and welding execution and technologies will be outlined. Inspections and tests conducted to assure quality during the manufacture of components will be described in Chap. 9.

8.1 Overview of Manufacturing Steps

In manufacturing a reactor vessel and other major nuclear components, strict selection of materials, execution of excellent welding technologies, and similar activities under elaborate design are conducted to secure high quality. Figure 8.1 shows the outline of these manufacturing steps. With steel manufacturers having achieved product quality improvement and development of manufacturing methods, such progress helps modern large-scale nuclear power plants secure high-quality steels as their structural materials. Steels are first delivered from steel manufacturers to machine manufacturers where the materials are put to predetermined assembly processes at well-equipped factories. Tests are conducted in each important step of the manufacturing process to check mechanical properties.

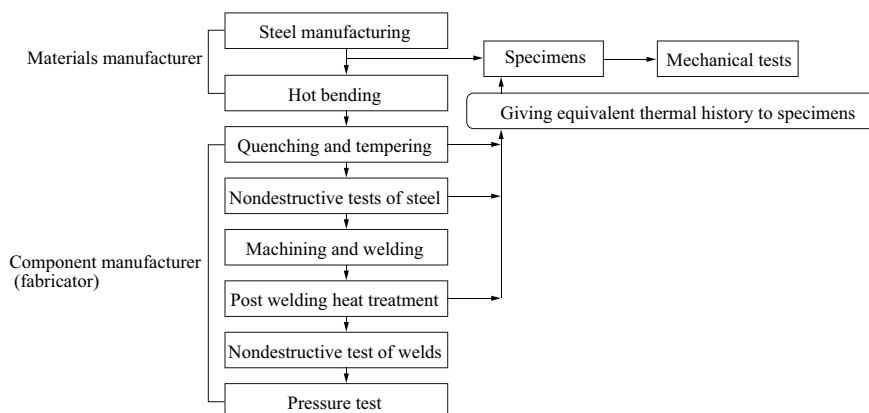


Fig. 8.1 Outline of manufacturing steps of reactor components (example of manufacturing from steel plate)

8.2 Selection of Materials

8.2.1 Concept Behind Selection of Materials

Materials used in important reactor structural components like a reactor (pressure) vessel, and reactor coolant system piping are required to have the following properties to confine reactor coolant and to secure structural integrity from the viewpoint of safety:

- (1) Minimum internal flaws and excellent uniformity.
- (2) Excellent mechanical properties and strength characteristics.
- (3) High fracture toughness.
- (4) Excellent weldability and workability.
- (5) Excellent corrosion resistance, resistance to stress corrosion cracking, etc.

Furthermore, such characteristics as lower content of elements causing induced radioactivity (for reactor pressure vessels and core internals), lower neutron irradiation embrittlement susceptibility (reactor pressure vessels, etc.), and smaller neutron absorption cross sections (core internals, etc.) are necessary depending on the situation of each component. Table 8.1 shows major reactor structural materials.

8.2.2 Manufacturing History of Materials

In the following, changes in light water reactor structural materials will be described using a reactor pressure vessel as an example, which is a very large structural component and the most important component from the safety viewpoint.

Table 8.1 Major reactor structural materials

	Component	Region	Type of material	Steel designation (example)	Remarks
Light water reactor	Reactor (pressure) vessel	Shell	Low alloy steel	SQV2A* (SA533B C1.1) SBV2B* (SA302B mod)	High-toughness, low irradiation embrittlement susceptibility
		Upper/lower head	Low alloy forged steel	SFVQ2A* (SA508 Gr.2C1.1) SFVQ1A* (SA508 Gr.3C1.1)	High-toughness, low irradiation embrittlement susceptibility
		Inner surface	Stainless steel weld overlay cladding	Type 308L, 309L	Prevention of corrosion of parts contacting cooling water
	Steam generator (PWR)	Shell, Upper/lower head	Low alloy steel	SQV2A* (SA533B C1.1) SBV2B* (SA533B C1.2)	High toughness
		Shell, tube sheet	Low alloy forged steel	SFVQ1A* (SA508 Gr.3C1.1) SFVQ1B* (SA508 Gr.3C1.2)	High toughness
		Upper/lower head	Low alloy forged steel	SFVQ1A* (SA508 Gr.3C1.1) SFVQ1B* (SA508 Gr.3C1.2)	High toughness
	Heat transfer tube		Ni-Cr-Fe alloy	NCF600, NCF690	Corrosion resistance, thermal conductivity
Fast breeder reactor	Recirculation system piping		Low-carbon austenitic stainless steel	Type 316L (nuclear grade)	Low stress corrosion cracking susceptibility
	Containment		Steel sheet Prestressed concrete	SGV480*	
High-temperature gas-cooled reactor	Reactor vessel		Austenitic stainless steel	Type 304, 316FR	High-temperature strength characteristics, compatibility with liquid metal sodium
	Cooling system components and piping		Austenitic stainless steel Chromium molybdenum steel	Type 304, 316FR 2.25Cr-1Mo steel Mod.9Cr-1Mo steel	High-temperature strength characteristics, thermal expansion characteristics
	Steam generator		Chromium molybdenum steel	2.25Cr-1Mo steel Mod.9Cr-1Mo steel	High-temperature strength characteristics, corrosion resistance
	Reactor vessel		Chromium molybdenum steel	2.25Cr-1Mo steel	High-temperature strength characteristics
High-temperature gas-cooled reactor	Intermediate heat exchanger heat transfer tube		Nickel-base alloy	Hastelloy XR	High-temperature strength characteristics

*: JIS designation

In JPDR, Japan's first light water nuclear power reactor, advanced Mn-Mo steel (commonly referred to as A302B mod) was used as steel for the reactor pressure vessel. On the basis of low-alloy steel ASTM A302B (Mn-Mo steel), which had been commercialized from an early period in the United States, 0.40 to 0.70 wt% Ni was added with the aim of improving low-temperature fracture toughness. Its plate thickness was relatively thin, i.e., 72–76 mm, and it underwent normalizing and tempering as heat treatments. The contents of P and S, as impurities in this steel, were about 0.011 to 0.013 wt%, relatively high when compared with modern steels.

Later, an increase in reactor size leads to thicker walls of reactor pressure vessels exceeding 100 mm, which in turn made it impossible to secure toughness only by adding Ni. As a result, water quenching and tempering came to be adopted as heat treatment methods. A302B mod steel (in plate forms) and A336 mod steel (in forged forms) treated by water quenching and tempering were used in the reactor pressure vessel of Tsuruga Power Station Unit 1. In later light water reactor pressure vessels, ASTM A533B C1.1, which is a Mn-Mo-Ni steel plate equivalent to the material above and treated by quenching and tempering, and A508 Cl.2, which is a Ni-Cr-Mo forged steel, were used. The latter was later changed to A508 Cl.3, which is a Mn-Mo-Ni forged steel.

In 1972, a revision was made in the United States to introduce linear fracture mechanics, as a safety assessment method on prevention of brittle fracture, into Sec. III, Appendix G of the ASME boiler and pressure vessel code (referred to as the ASME code in the following). This practically established Sec. III of the ASME code as a design basis to prevent both the brittle fracture and fatigue fracture of pressure vessels. Furthermore in 1973, Sec. XI, Appendix A of the ASME code and NRC Federal Regulations 10 CFR Part 50 were established to secure reactor safety, providing the foundation for techniques to assess fracture toughness required for materials used. Since this revision, the concept of reference temperature RT_{NDT} determined by the drop weight test and the Charpy impact test has been taken up as a design concept to address the brittle fracture of reactor pressure vessels, and this concept is now widespread in the world.

In addition, it was made clear around 1970 that neutron irradiation during the in-service period causes a reduction in fracture toughness proportional to impurity content in steel (so-called neutron irradiation embrittlement) in the reactor vessel core region. For this reason, standard values considering a reduction in toughness due to neutron irradiation embrittlement were specified, while regulations on impurity elements like Cu and P that accelerate irradiation embrittlement were intensified along with stricter requirements on materials. Table 8.2 shows an example of impurity regulation on the ASME SA 533B Cl.1 steel (equivalent to the A533B Cl.1 steel in the ASTM standard). In 1974, specific limits on the amounts of Cu and P in SA533B Cl. 1 steel plates applied to core regions were added with the aim of mitigating irradiation embrittlement. At that time, steel plates were used in core regions and the use of forged steel was limited to nozzles and flanges. Later, technology to manufacture large forged steel products was developed from the viewpoint of reducing in-service inspection of welds and shortening the construction period. This technology was applied to reactor pressure vessels built after the 1980s. In order to improve the quality of products using increasingly large ingots, strict limits on composition, including reduction of impurity elements, have been added, as described later.

Meanwhile, materials for fast breeder reactors need to be selected from those having strength at high temperatures where creep may occur, compatibility with liquid metal sodium used as coolant, and excellent resistance against thermal load, which is the principal load. On the basis of the above viewpoint, austenitic stainless steel is used in key components like reactor vessels, while chromium-molybdenum steel is used in steam generators for which corrosion resistance and heat transfer

Table 8.2 Upper limits of impurity contents for steel used in the core region of light water reactor pressure vessels (in wt%)

Time	Steel plate SA533B Class 1			Forged steel SA508 Class 2 and Class 3			Remarks
	Cu	P	S	Cu	P	S	
1974	0.10	0.012	0.035	-	-	-	Upper box: molten steel (Ladle) analysis
	0.12	0.017	0.035	-	-	-	
1983	0.10	0.012	0.015	0.10	0.012	0.015	Lower box: product analysis
	0.12	0.015	0.018	0.12	0.015	0.018	
Typical specification currently required	0.05	0.008	0.004	0.05	0.008	0.004	

capability are required. The reactor vessel of Monju, constructed as a fast breeder reactor, was made of Type 304 stainless steel, with its steam generator made of 2.25Cr-1Mo steel. Later, improvements were made in austenitic stainless steel for fast reactors, and Type 316FR stainless steel was developed. In this steel, carbon content was minimized to suppress the production of chromium carbides, and nitrogen was added to secure the required strength. Regarding chromium-molybdenum steel, the application of high chromium steel was studied for its thermal stress durability due to excellent strength at high temperatures and a low coefficient of thermal expansion. In addition, Mod.9Cr-1Mo steel and 12Cr steel were developed, which were expected to be applicable to future reactors. The latter steel overcame its previous weak point of ductility by means of steel purity control.

For high-temperature gas-cooled reactors, as is the case with fast breeder reactors, materials are selected in consideration of strength and creep characteristics at high temperatures in preparation for their use at high temperatures. The High-Temperature Engineering Test Reactor (HTTR) of the Japan Atomic Energy Agency uses, in its reactor pressure vessel, 2.25Cr-1Mo steel, which is superior to Mn-Ni-Mo steel used in light water reactors in terms of high-temperature strength and, in particular, creep characteristics. In the HTTR, this 2.25Cr-1Mo steel was put to heat treatment of normalizing and tempering (NT) to enhance strength and toughness. Furthermore, the chemical composition of steel (Si, Mn, P, Sn, Sb, As, and Co) is limited to the range specified in the related Japan Industrial Standard (JIS) provisions (SCMV4-2) with the aim of suppressing not only the decrease in the fracture toughness due to neutron irradiation embrittlement and thermal aging but also activation. With the heat transfer tubes of the intermediate heat exchanger of the HTTR having the possibility of reaching 900 °C at maximum under normal operation and 950 °C in an accident, Hastelloy XR, a nickel-base corrosion and heat-resistant alloy, is used. On the basis of Hastelloy X, which has high strength at high temperatures and excels in corrosion resistance, Hastelloy XR has been developed as a result of increasing the Mn content, optimizing the Si content, and decreasing the Al, Ti, and Co contents.

8.3 Steel Manufacturing Technology

8.3.1 *Improvement of Quality of Material Interior*

In order to improve material interior, it is important to take the following measures including reduction of non-metallic inclusions and hydrogen content, and measures against shrinkage pores and segregation.

- Sulfur and oxygen and hence non-metallic inclusions are reduced by means of reduction in sulfur content through molten iron desulfurization and the control of vacuum degasification condition.
- Dehydrogenation treatment is performed by means of reduction in steel-dissolved hydrogen, dehydrogenation annealing, slow cooling after rolling, and other means.
- Rolling is performed after crimping shrinkage pores generated inside the steel ingot by using a large-capacity press.
- Compositional variations in a product are minimized by cutting off the head and bottom of a steel ingot where segregation exists, and the mold shape is optimized.

8.3.2 *Securing of High Toughness and Excellent Material Characteristics*

The following are some of the ideas worked out in manufacturing steel to obtain high-toughness and excellent material characteristics that form the basic conditions for the design of steels used in major nuclear reactor components.

- To enhance strength by increasing lattice distortion through an increase in the concentration of alloy elements added into the crystal lattice as substitutional or interstitial elements in a solid solution (a homogeneous phase in which atoms of another metal or non-metal are perfectly incorporated into the crystal lattice).
- When alloy elements gather to form aggregate or precipitate, the movement of dislocations is prevented and the strength increases. To accelerate precipitation, normally alloy elements are kept at high temperatures long enough to form a solid solution, which is quenched and put to aging at temperatures below the solubility limit.
- Crystal grains are made finer to increase the density of grain boundaries and obtain strength. For this purpose, methods such as heat treatment during which quenching is conducted in the transformation temperature zone and the addition of adequate quantities of aluminum, niobium, vanadium, and other elements into steel are used to precipitate carbides and nitrides in fine form with the aim of suppressing the growth of crystal grains under high-temperature heating and making such precipitates act as cores for transformation during cooling.

- Work hardening is produced by adding plastic deformation. The work hardening increases the yield strength and tensile strength easily by cold working, but requires caution because it affects toughness and ductility adversely.
- A clean texture with reduced segregation and heterogeneity of alloy elements is obtained by reducing impurity elements (P, S, Cu, As, Sn, and Sb) and inclusions.
- Ni as an additive element is necessary to improve hardenability and secure toughness and strength. It is necessary to control the C content within an adequate range from the viewpoint of the balance between strength and toughness. It is necessary to limit the addition of Al and N within the range required by grain refining because an excessive addition degrades toughness.
- Cr and Mo improve strength at normal and high temperatures but increase the susceptibility to under-clad cracking. For this reason, the addition of Cr and Mo is adjusted so as to make the ΔG value ($= \text{Cr} + 3.3\text{Mo} + 8.1 \text{ V} - 2$) negative, while V is kept at a low level by using a high-purity molten iron.
- It is necessary to minimize the neutron irradiation embrittlement of reactor pressure vessel steel to prevent brittle fracture. Since Cu and P are harmful as impurity elements, it is necessary to reduce Cu and P contents by using high-purity hot pig iron and low-phosphor steel ingots.

8.3.3 Steel Manufacturing Processes

Steels used in components are manufactured through such processes as iron making to make pig iron in a blast furnace, steelmaking to make steel from pig iron in a steelmaking furnace, rolling to shape various products in their final forms, and surface treatment. In the following, a typical process of manufacturing steel for reactor pressure vessels is described.

- (1) Iron making: Put iron ores (oxidized iron ores), gas coke (fuel and reducing agent), and other materials into a blast furnace, and blow high-temperature air to produce pig iron. Because of its very high carbon content, pig iron serves as a raw material for steel and cast iron. As a preliminary treatment to reduce impurities, sintered ores, CaO, and lime are blown into molten pig iron.
- (2) Steelmaking: In a typical method of steelmaking using a converter, carbon and impurities in molten pig iron are eliminated by burning and oxidation. Then, the molten steel is moved into a ladle to add Fe–Mn, Al, and other elements, with alloy components added and adjustments carried out to obtain the intended composition. Gases such as hydrogen and oxygen are removed by degassing. The melt is poured into a casting mold in an oxidation-free condition using Ar gas, and the melt left to solidify into an ingot.
- (3) Commercial production: In the case of rolling, steel ingots obtained after the steelmaking process are heated, forged, and rolled to refine grains. Then, they go through hot working to provide high toughness and heat treatment after forging and rolling. Large-capacity presses are used to forge large-sized products. After

this step, steel plates intended for use in the core region are quenched and tempered as heat treatment.

- (4) Inspection: After heat treatment, specimens taken out from steel plates are put to quenching, tempering, and post weld heat treatment (PWHT), and are subjected to tensile, impact, and drop weight testing. In addition, analysis of chemical components of products, defect inspection, and dimensional inspection are conducted.

8.4 Heat Treatment

Reactor pressure vessel steel is put to heat treatment such as refining (quenching and tempering) to increase toughness and welding for component assembly. Then, post weld heat treatment (PWHT) is conducted for ferritic steel. PWHT will be described in Sect. 8.7.

8.4.1 *Types of Heat Treatment*

Heat treatment is a combination of heating and cooling to give desired properties to steel, and is roughly classified into quenching, normalizing, annealing, and tempering.

- (1) Normalizing: After heating steel into the austenite domain, steel is cooled in the air. Being the simplest heat treatment to adjust the structure, this process improves mechanical properties by downsizing crystal grains and standardizing the structure.
- (2) Quenching: After heating steel into the austenite domain, it is cooled down rapidly by using water or oil. Quenching makes steel very hard. After being cooled down rapidly, steel is cooled down slowly below the point at which the transformation from austenite to martensite starts in order to prevent hot cracking. After quenching, tempering provides excellent toughness and fatigue strength to steel.
- (3) Tempering: After quenching or normalizing, steel is heated to an adequate temperature again and then cooled down slowly. Tempering steel at about 600 °C to recover toughness is called thermal refining.
- (4) Annealing: After heating steel into the austenite domain and keeping it there, it is cooled down slowly. This heat treatment removes strains produced inside by work hardening, softens the structure, and improves ductility. This includes full annealing, isothermal annealing, and stress relief annealing.

8.4.2 *Securing of Toughness*

In order for Mn-Ni-Mo steel used in reactor pressure vessels to secure satisfactory strength and high toughness, the steel must be cooled down from the austenitizing temperature (about 900 °C) at a somewhat higher cooling rate. This is difficult to achieve in extremely thick steels by normalizing (radiation cooling in the air); to achieve this, rapid cooling in water (with forced circulation) is conducted to secure the desired hardenability. Large-sized forged steel products are machined into close to their final shapes before conducting refining heat treatment, and measures are taken to keep the temperature inside the wall and the cooling rate as homogeneous as possible.

A higher quenching temperature causes crystal grains to become too large; for this reason, a temperature around 880 °C is selected in consideration of suppressing grain growth.

Tempering steel at too high a heating temperature or for too long a duration causes crystal grains and precipitates to become coarse, which in turn degrades toughness. In order to avoid this, an adequate combination of the heating temperature and duration is selected with attention paid to secure sufficient toughness (about 640 °C).

8.5 Welding Procedure Control and Welding Materials

8.5.1 *Rules and Standards for Welding in Japan*

The Japan Society of Mechanical Engineers (JSME) has been working on the formulation of private sector standards in response to the government's policy that technical standards for nuclear power plant components should be performance-based and private sector standards should be utilized. As for the welding of components, the Rules on Welding for Nuclear Power Plants was published in 2001 in accordance with the performance specification of the Ministerial Ordinance. This rules on welding was established based on the interpretation of the Ministerial Ordinance on the Technical Standard for Welding of Electric Facilities, by referring to the relevant ASME codes at that time, coordinating with the standard establishment work of other organizations in Japan, and adding its own considerations. The content of the rules for welding is divided into four parts: Part 1 is the rules for welding, Part 2 is the rules for welding practice certification, Part 3 is the rules for welder's skill certification, and Part 4 is the commentary. The rules on welding was revised in 2007 to reflect the revision of the rules for welder's skill certification, the Rules on Design and Construction for Nuclear Power Plants, and related Japanese Industrial Standards (JIS). Since then, the Rules on Welding for Nuclear Power Plants has been revised by the 2012 edition, 2016 edition, and their supplements.

Regarding welding on electric workpieces in government regulation as mentioned above, the Electric Utilities Industry Law made it obligatory to confirm the compliance of welded structures with the welding technical standards. The compliance with the ministerial ordinance (the Ordinance of Ministry of International Trade and Industry No. 123 in 2000) that stipulates technical standards for welding on electrical workpieces was a precondition for acceptance of utility's welding self-inspection. This ministerial ordinance was intended to turn the past Ordinance of Ministry of International Trade and Industry No. 81 into performance-based criteria. However, this ministerial ordinance was abolished in January 2006, and the compliance was put under the rules of Paragraph 15, Section 9, Ministerial Ordinance on Technical Standards for Nuclear Power Generation Facilities (Ordinance of Ministry of Economy Trade and Industry No. 62). As the conditions to meet these rules, an interpretation of the rules established in December 2005, and a partly revised interpretation in December 2007 were applied as of 2008.

After the enactment of the new regulatory standards by the Nuclear Regulation Authority (NRA), the aforementioned laws, ministerial ordinances, and their interpretations have been abolished and the new laws, rules, and interpretations of the rules have been established. In other words, Article 43-3-13 of the Act on Regulation of Nuclear Source Materials, Nuclear Fuel Materials, and Reactors stipulates that welding shall be confirmed to conform to the technical standards in the welding inspection by licensee. These technical standards are included in the Regulations Concerning Technical Standards for Commercial Power Reactors and Their Auxiliary Facilities, and the Article 17, Item 15 of the Regulations stipulates the following four criteria: A: no discontinuity or peculiar shape, B: no risk of cracking due to welding, confirmed by nondestructive examination, C: appropriate strength, D: welded by a welding method, welding equipment and welder confirmed to be appropriate.

In the interpretation of the standards, the concept of welds of main pressure resistance parts and the concept of A, B, and C among the aforementioned four criteria are described in accordance with the provisions of Article 17, Item 15 above. In addition, this interpretation states that welds that conform to the provisions of the relevant regulations shall be in accordance with the provisions of the Rules on Welding 2012 Edition (including 2013 Supplement) and the Rules on Design and Construction 2012 Edition (including 2013 Supplement), with additional requirements indicated during the technical evaluation for the endorsement of the Rules by NRA.

The NRA will continue to respond to the use of periodically revised private sector standards in its regulatory standards, and the technical evaluation of standards related to welding will be conducted considering the status of revisions of the Rules on Welding and the Rules on Design and Construction that are referred to in the interpretation of technical standards.

8.5.2 *Items to Consider*

For nuclear facilities, the prevention of weld flaws and the securing of corrosion resistance and toughness of welds are controlled strictly on the whole. The following are the items taken into consideration from the viewpoints of welding materials and procedures:

- A progress in the manufacturing technology related to the securing of toughness of reactor pressure vessel steels was accompanied by an improvement in the performance of welding materials. Efforts have been made to improve the mechanical properties of submerged arc welding materials, coated electrodes, TIG electrodes, etc. Since toughness is affected by welding conditions and heat treatment, procedural control is also important.
- Parts exposed to high temperatures need measures against general corrosion, and corrosion-resistant cladding or corrosion-resistant alloys are used in parts requiring such measures. In addition, piping needs measures against stress corrosion cracking (SCC).
- Introduction of hydrogen into weld metal during welding operation may cause delayed cracking. This is caused by the transformation of diffusible hydrogen remaining in the weld metal into hydrogen molecules at grain boundaries. This occurs in regions with high residual welding stress, with the hydrogen content tending to increase near the surface. For this reason, this phenomenon requires caution. In order to cope with this problem, it is necessary to lessen the effect by using low-hydrogen-content materials and implementing pre- and post-heating control during the welding work.
- Hot cracking tends to occur in the weld overlay cladding of stainless steel and Inconel-based alloys on the inner surface of a vessel. To address this problem, the quantity of ferrite in the weld metal is controlled and the quantities of impurities like P and S that affect hot cracking are minimized, while the welding heat input is suppressed.
- When a heat-affected zone is exposed to high temperatures in a post-welding heat treatment, reheat cracks occur along the grain boundaries whose strength has lowered. This is known as under-clad cracking in a build-up heat-affected zone on the inner surface of a reactor pressure vessel. Measures such as base metal improvement, multi-layer welding, and the avoidance of large heat input have been taken.

Detailed structures of welds as well as “flaws” and “defects” likely to occur as a result of welding will be given in Sect. 9.2.2 of Chap. 9, together with the description of weld flaws.

8.5.3 *Welding Materials*

Best-suited welding materials are selected depending on the base metal and performance requirements. Although materials manufactured according to JIS standards are commonly used, special requirements may be added as nuclear specifications. The following are typical JIS standards for welding materials:

- JIS Z3321: Stainless steel rods, wires, and strip electrodes for welding.
- JIS Z3323: Stainless steel flux cored wires and rods for arc welding.
- JIS Z3351: Solid wires for submerged arc welding of carbon steel and low-alloy steel.
- JIS Z3352: Fluxes for submerged arc welding and electroslog welding.
- JIS Z3224: Nickel and nickel alloy covered electrodes.

JIS standards are not established for welding materials for Mn-Mo-Ni steel used in reactor pressure vessels, while materials that meet performance requirements are developed by manufacturers. As is the case with base metals, these materials need consideration of low-temperature toughness and neutron irradiation embrittlement. For this reason, the quantities of Cu and P as chemical compositions are suppressed compared with those of ordinary welding materials.

Weld overlay cladding with stainless steel or nickel alloy is provided on the inner surface of a reactor pressure vessel to secure corrosion resistance. Strip electrode overlay cladding is in common use as an overlay welding method from the viewpoint of efficiency and corrosion resistance. In addition to this method, TIG welding and, recently, plasma welding are used.

Materials such as Type 308L, 316L, and 308NbL are used as welding materials for austenitic stainless steels. Weld metals are designed to contain 5 to 15% of the δ ferrite phase to prevent hot cracks. Low-carbon stainless welding materials and Co quantities are limited strictly to prevent stress corrosion cracks and to suppress activation.

8.6 *Welding Technology*

Welding is a mainstay technology in the process of assembling reactor structures. Although a variety of welding methods are available, most nuclear components are welded by the most widely used arc welding method.

In arc welding, an arc is generated between the base metal and the electrode or between two electrodes, and the arc heat generated is used to melt joints to weld them. High-temperature heat emitted from an arc causes the weld metal to be affected adversely by oxidation or nitrification with the presence of O_2 or N_2 in the atmosphere. To prevent this and secure the quality of welded joints, an arc is protected and shielded from the atmosphere. Depending on how this can be done, several types of welding are available.

The types of welding used for nuclear components are the shielded metal arc welding method, the submerged arc method, the metal inert gas (MIG) method, and the tungsten inert gas (TIG) method, all of which come under the arc welding processes.

Methods of welding other than the arc welding processes are the electroslag welding method, the electron beam welding method, and the laser welding method, though they are used in part.

In addition, austenitic stainless steel and nickel alloy are overlay-welded by the strip electrode overlay welding method to secure corrosion resistance against high-temperature, high-pressure water on the inner surface of a reactor vessel. In the following, major welding methods are described.

8.6.1 Shielded Metal Arc Welding (SMAW)

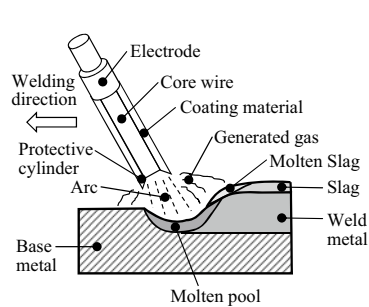
A high voltage is applied between a covered electrode, which is a core wire coated with flux, and the object to be welded to generate an arc, and the heat from the arc is used for welding (Fig. 8.2a). Heat from the arc causes the tip of the electrode to be melted and moved to the base material, and the molten electrode merges with a part of the base metal to form a molten pool. A part of the coating material turns into gas to cover the arc, while another turns into molten slag and dissolves into the molten metal. The weld is protected from the atmosphere mainly by the gas and slag generated from the coating material. This is a general-purpose welding method mainly operated manually.

8.6.2 Submerged Arc Welding (SAW)

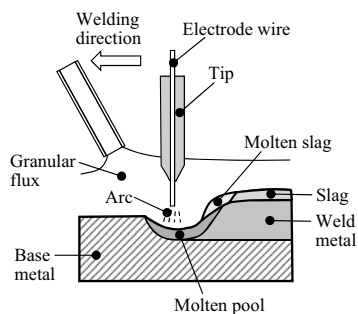
In this method, granular flux is built up on the surface of a joint beforehand, and an electrode wire is inserted into the flux to produce an arc between the base metal and the wire in the flux to achieve continuous welding. During the welding operation, molten slag and gas generated from the flux melted and decomposed by arc heat cover the molten pool to shut off the air (Fig. 8.2b).

Normally, this method uses a large current and hence is efficient, allowing welding conditions to be kept constant; this results in homogeneous welds of high quality because of the metallurgical effect of the slag. The welding position is normally in a downward direction.

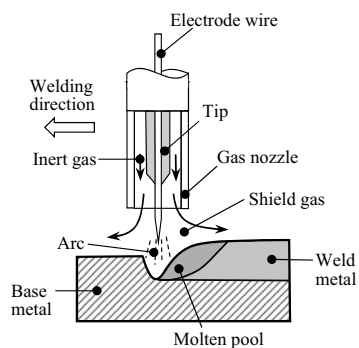
This welding method is suitable to the welding of thick-wall structures with long, straight weld lines such as steel pipes, pressure vessels, and tanks. This method has been in use as a welding procedure for butt joints in thick-wall structures such as reactor pressure vessels.



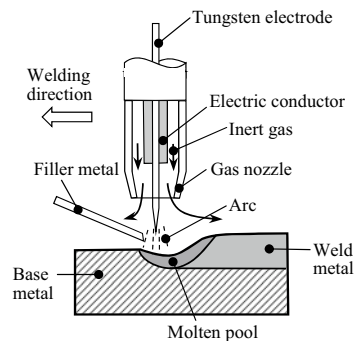
(a) Shielded metal arc welding method



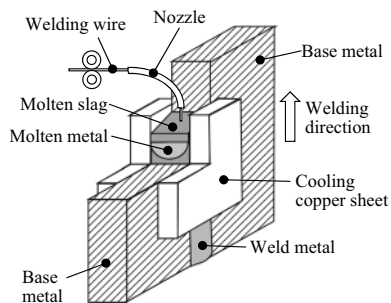
(b) Submerged arc welding method



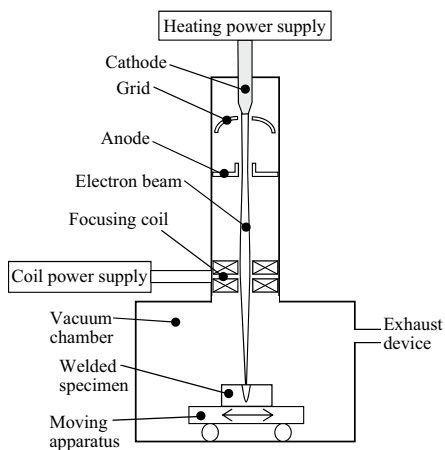
(c) MIG welding method



(d) TIG welding method



(e) Electroslag welding method



(f) Electron beam welding method

Fig. 8.2 Schematic images of major welding methods (these figures were drawn based on Reference 8)

8.6.3 *Metal Inert Gas Welding (MIG)*

As is the case with the TIG welding described in the next section, this method is based on the inert gas-shielded arc welding method in which a metal arc is covered by flowing inert gases such as argon and helium (Fig. 8.2c).

A wire that forms filler materials is fed continuously in an inert gas atmosphere to generate an arc between the wire and the base metal for welding. Depending on the type of material forming the object to be welded, argon, helium, or a mixed gas consisting of such an inert gas and O_2 or CO_2 are used. In the MIG welding and the above-mentioned shielded metal arc welding and submerged arc welding, the electrode itself produces an arc and is melted. These methods are used in joint welding with plates having a wide thickness range of about 1.5–25 mm.

The narrow gap MIG welding method is used for grooves with their width narrowed to 20–30 mm and can be used for thick-plate welding as well. The advantages of this method in general are as follows: (1) with a smaller welding heat input and hence a smaller heat effect, the toughness of a weld is excellent and the width of the welding heat-affected zone is small; (2) with the groove being narrow, the quantity of weld metal and weld strain is small; and (3) the extremely small quantity of diffusive hydrogen in the weld metal makes it difficult for delayed cracking after welding to occur.

The narrow gap MIG welding method is applied to important weld joints in nuclear components such as reactor pressure vessels in view of improving quality and welding efficiency.

8.6.4 *Tungsten Inert Gas (TIG) Welding*

An arc is generated between a tungsten electrode and a base metal in an inert gas atmosphere, and the heat from the arc is used to melt the filler metal and the base metal to weld them (Fig. 8.2d). This method allows welding in all positions easily and is used widely for welding thin plates and pipes. This method allows beads to be formed easily on the backside in first-layer welding and is therefore often used for one-side welding. Characterized by a low O_2 content in the weld metal, easiness in adjustment of weld penetration and deposition, high quality, and high efficiency, TIG welding is applied widely as an excellent method.

The pulse TIG welding method uses periodic pulse welding current. This method may be provided with a pulse current control capability to facilitate arc generation and improve weld penetration adjustment and arc stability.

8.6.5 Electroslag Welding

The difference between the arc welding method and the electroslag welding method is that the former uses arc heat, whereas the latter uses the heat generated by the resistance of molten slag to melt the welding wire and the base metal. The welding wire that forms the electrode is placed in the space surrounded by the end face of the base metal and the water-cooled metal plate, with the space filled with flux. In the early stage after the start of welding, an arc is generated between the wire and the base metal. However, as the surrounding flux is heated and melted by the heat from the arc to form molten slag, the heat source shifts to the heat generated by electric resistance. The molten slag heated to a high temperature by its own resistance melts the continuously fed wire and the base metal, and as a result, molten metal is formed at the bottom of the molten slag (Fig. 8.2e).

The electroslag welding method is characterized by the absence of spatter during welding, high thermal efficiency with welding efficiency close to 100%, and easy preparation of the grooves, as the grooves can be cut without gas cutting.

8.6.6 Electron-Beam Welding (EBW)

A convergence coil is used to focus thermoelectrons (to form electron beams), accelerated to high speed in a high vacuum (normally 10^{-4} to 10^{-6} Torr, or 10^{-2} to 10^{-4} Pa), and make them collide onto the welding wire to change the dynamic energy of the electrons into thermal energy for melting and welding (Fig. 8.2f). The following are the features of electron beams, but this method is subject to cost restrictions because it can be used only in a vacuum.

- (1) With high energy density, one-pass welding can be applied even on a thick plate. This reduces the quantity of welding input heat, allowing high-precision welding with less weld deformation.
- (2) Welding is usually executed in a high vacuum; this leaves weld metal highly clean and results in a high-quality weld metal.
- (3) Because of the superb heat concentration, very deep weld penetration and narrow weld metal width are achieved.
- (4) Welding operation is highly reproducible and reliable.

8.7 Post Welding Heat Treatment

Post welding heat treatment (PWHT) is intended mainly for removing residual stress, improving the mechanical properties of welds, stabilizing weld metal structures, and removing residual hydrogen. The effect of PWHT is remarkable in low-alloy heat-resisting steels and thick-plate welds, and the execution of PWHT is obligatory.

In Mn-Mo-Ni steel for reactor pressure vessels, PWHT is usually carried out at temperatures ranging from about 600 to 650 °C. Caution is required because this temperature range is close to that where sigma phase embrittlement progresses in austenitic stainless steels and their weld metals.

It is known that welding materials used in the welding of Mn-Mo-Ni steel may exhibit reductions in 0.2% proof stress and tensile strength when exposed to high temperatures for a long time period.

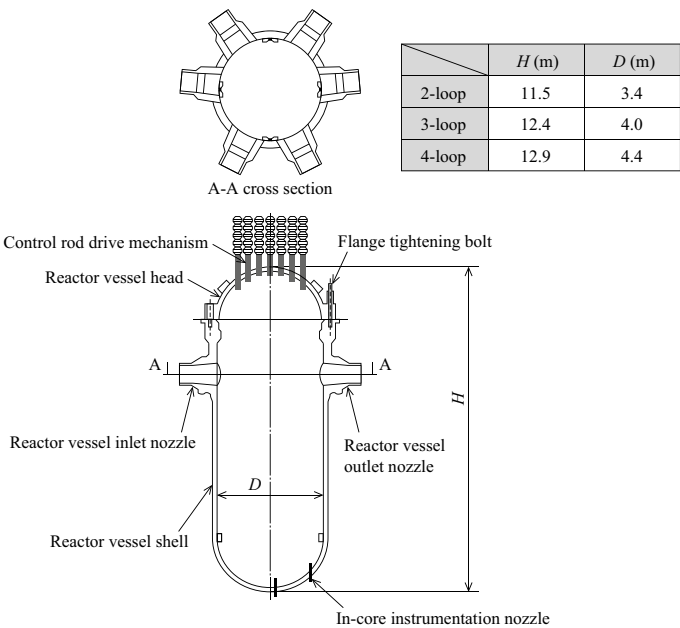
In weld overlay cladding on the inner surface of a pressure vessel, 308L or 347-based austenitic stainless steels or nickel alloys are used as described before. They are affected by PWHT if they have high ferrite contents, and their Charpy absorbed energy is reduced when exposed to high temperatures for a long time period.

8.8 Examples of Component Manufacturing

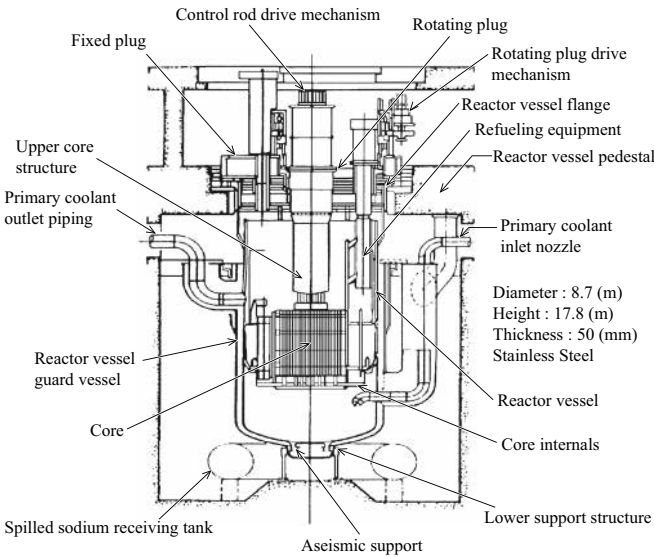
The conventional manufacturing method of light water reactor pressure vessels uses bending of thick steel plates and axial welding to manufacture shells. As described in Sect. 8.2.2, however, recent progress in technologies to manufacture large forgings enables vessels to be manufactured with reduced weld lines. This leads to the prevention of occurrence of non-ductile fracture originating from flaws that might occur in weld lines. At the same time, this facilitates manufacturing and reduces the number of locations requiring in-service inspection, contributing greatly to the improvement of reactor operation efficiency and reduction in radiation exposure. In an up-to-date manufacturing method based on the use of large forgings, a substantial reduction in the number of weld lines requiring inspection is achieved because of the capability to manufacture the core region and the upper and lower heads as single-piece structures.

Figure 8.3 compares typical reactor vessel structures and materials between a light water reactor (PWR) and a fast breeder reactor. The reactor vessel and main coolant system components of a light water reactor are made of thick-wall steel, with their principal load being pressure and seismic force. In contrast, the reactor vessel and main coolant system components of a fast breeder reactor operated under low pressure have thin-wall structures favorable to withstand thermal stress, which is the principal load.

In the assembly work of the large thin-wall reactor vessel of the fast breeder reactor Monju, the vertical assembly method was used to avoid deformation arising from laying the reactor vessel horizontally. In addition, since austenitic stainless steels with large coefficients of thermal expansion were handled, machining was carried out in a constant-temperature room. For the welding of major parts, the narrow gap hot TIG welding method, characterized by low heat input and low strain, was adopted, and deformation was minimized by conducting simultaneous welding using multiple torches.



(a) An example of pressurized water reactor vessel (3-loop plant)
(Translated from Fig. 3.4.2 in reference 9)



(b) Fast breeder reactor Monju
(Translated from Fig. 5-1-15 in reference 4)

Fig. 8.3 Typical reactor vessel structures

Exercises

1. Enumerate matters to be kept in mind for welding and the purposes of post-welding heat treatment.
2. Taking up a reactor pressure vessel as an example, describe typical steel manufacturing processes. Also summarize the history of manufacturing technologies in improving the quality of steel.
3. Enumerate ideas worked out to ensure superb material properties in steel manufacturing.

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Chapter 9

Examination



To guarantee that components are of quality high enough to assure structural integrity, the material properties and potential “flaws” (or defects) (the definition of these terms are described in Sect. 9.1) must meet the specific requirements in each stage of the production and forming of the materials, and of the manufacturing, transportation, assembling and installation of the components, which were described in Chap. 8, as well as at the design stage the components should meet the requirements on design margins as mentioned in Chaps. 5 and 6. Therefore, it is obvious that the examinations and testing for checking that those requirements are met play a key role in quality assurance. In particular, if a detrimental flaw, or more specifically a defect, is present in a component, this may cause damage to components or could lead to fracture of components in some cases. As means to detect such defects, pressure testing and leak testing for checking the pressure resistance and leak tightness of components, respectively, have long been performed. Nondestructive examination for detecting defects in a nondestructive manner has also become essential. In general, the term “examination” is used in a broad sense to include performance examination and mechanical testing. This chapter describes only the examinations designed for defect detection from the viewpoint of structural integrity.

“Flaws and defects” occur in various forms. Meanwhile, you have learned in this book that component failure also occurs in various modes. The causal relationship between a particular form of flaw/defect or material property and a particular failure mode, and the resultant relationship between the failure and a particular defect, vary with failure modes. Therefore, to assure quality, it is important to understand what forms of flaw/defect must be specified for each failure mode and select an examination method that enables you to detect the flaw/defect with sufficient reliability.

To assure a sufficient level of quality, it is necessary to consider that the quality level required for a particular component varies depending on functions of the system or of the component itself. In other words, the importance of performance or function, such as safety, that the system or component must fulfill varies according to the system or component. Concerning structural codes and standards, Chap. 5 stated that

categorization, or component classification, was practiced. In addition, in the case of pressure-retaining components, the circumferential stress caused by an internal pressure in the cylindrical shell is approximately twice the axial stress. Since the fragility of such components varies according to the orientation of the flaw, welded joints are also classified.

Summarizing the above, the quality level that the component is required to achieve varies from component to component, or even from part to part, depending on the severity of failure modes to be assumed, the shape and size of flaw or defect (flaw/defect) to be detected, the importance of performance or function to be achieved, and so on. An appropriate method for detecting the required flaw/defect must be carefully selected in consideration of shape and dimensions of flaw/defect and its reliability. This chapter first introduces, in Sect. 9.1, important examination-related terms that need to be defined. Then it describes to what forms of flaws and defects attention should be paid for each failure mode in Sect. 9.2. Section 9.3 describes nondestructive examination (or testing) methods that are appropriate for detecting flaws and defects of each form. Section 9.4 describes what types of inspection (testing) methods should be used according to the importance levels of components (component classification) under the existing structural codes and standards. Section 9.5 discusses pressure testing and leak testing conducted from the viewpoint of ensuring pressure resistance and leak tightness. Section 9.6 describes the trends of requirements on the skill of inspectors in terms of certification.

As mentioned in Item (1), Sect. 1.5.2, Chap. 1, the safety regulation was drastically changed in response to the accident at the Fukushima Daiichi nuclear power station that happened in March 2011 (the Fukushima Daiichi nuclear accident), and the related safety measures were newly regulated or strengthened, including requirements for installation of facilities for coping with severe accidents, etc. In parallel with the safety regulation changes, the new inspection systems started in 2020, as described in the same Item (5), in order to clarify that the prime responsibility for ensuring the nuclear safety lies with the owner (or licensee), as a result of reflecting the recommendations made by the International Atomic Energy Agency (IAEA) through the Integrated Regulatory Review Service (IRRS). The final section of this chapter, Sect. 9.7, discusses the concepts of examinations for facilities for coping with severe accidents, etc., and further the details of changes in the new examination and inspection systems, with a focus on the nondestructive examinations and inspections for welds. Defects in welds, i.e., weld defects, in particular, those that were detected by examination or inspection, are basically required to remove. The details of the regulations for weld defects in the new regulation system are introduced in this section.

9.1 Definitions of Terms

Even terms we casually use everyday must be strictly defined if accurate judgment is required under structural codes and standards. For example, if an imperfection such as a cavity or flaw is found in a material, you are required to determine whether it is an “imperfection that allows the material to be used as is” or an “imperfection that does not allow the material to be used as is and must be removed.” In such a case, a strict definition is required as to what the term “flaw” means when used. In addition, if you assess the result of examination (testing, to be precise), it is important to distinguish between cases where the same result is drawn regardless of the testing engineer (e.g., a case where an indicated pressure drop is within the criterion), and cases where the assessment requires a high level of qualification and knowledge. In the latter case, the engineer would be required to objectively prove that his/her qualification, knowledge, etc., are appropriate. Based on this perspective, this section provides strict definitions of those terms that are frequently used in examination and testing and that we ambiguously use everyday. The following definitions are provided in the ASME Boiler and Pressure Vessel Code [1], etc.

- **Testing:** An element of verification for determination of the capability of an item to meet specified requirements by subjecting the item to a set of physical, chemical, environmental, or operating conditions [1].
- **Examination:** Specific actions by qualified personnel using qualified procedures to verify that items, fabrication processes, and pre-service requirements are in conformance with specified requirements [1].
- **Flaw:** An imperfection or discontinuity that may be detected by nondestructive testing and is not necessarily rejectable [2].

The Japanese Industrial Standards [3] define “imperfection” as “deviation from an ideal weld,” and defects indicated below as “unacceptable imperfections.” The meaning remains the same.

- **Defect:** One or more flaws whose aggregate size, shape, orientation, or properties do not meet specified acceptance criteria and are rejectable [2].

The difference between testing and examination can be summarized as follows. Testing means a method for checking whether or not the requirement is met, while examination may include assessment of the result of testing performed by qualified personnel if it is necessary for the personnel to assess whether the result meets the acceptance criterion.

9.2 Forms of Flaws/Defects and Failure Modes

9.2.1 *Forms of Flaws/Defects that Cause Various Failure Modes*

It can be said that fracture or failure would hardly occur if material selection, design, material production, and component manufacturing were performed in accordance with the provisions of the structural codes and standards. However, there seems to be no end to fracture or failure in reality. There have been many reports on causes of fracture or failure [4]. This section will reorganize failure, and study what mode of failure occurred and what specifically caused the failure.

In this chapter, fracture is defined as the situation of crack that the crack is extended to the majority of cross-sectional area of a component, and failure, including fracture, means loss of a function that a particular component must fulfill. Failure may occur in three categories: (1) a defect develops into a fracture; (2) a defect progresses and penetrates the component to cause leak; or (3) excessive deformation results in a functional loss (although there is a difference between failure that occurs under a single load application and under cyclic load application). This section focuses on flaws and defects and therefore discusses the first two failure categories. The failure modes in the non-creep temperature range described in Chap. 5, and the failure modes in the creep temperature range described in Chap. 6, can be summarized collectively as follows.

(1) Ductile Fracture

Possible causes include lack of thickness or strength due to inappropriate selection of a material, design error, inappropriate workmanship (e.g., occurrence of a weld defect) and aging degradation (e.g., thinning due to corrosion), and overload not postulated in the design. Any of these could cause excessive plastic deformation to the respective part, which may lead to void formation, growth, coalescence, etc., resulting in fracture. The following are examples of accidents caused by this mode of failure at nuclear power plants.

- In August 2004, the wall of the condensate piping in the turbine building of Unit 3 of the Mihama Power Plant of Kansai Electric Power Company was worn and thinned due to the flow behind an orifice, became unable to withstand the cooling water pressure, and was ductile-fractured. In this accident, five workers died and six workers were seriously or slightly injured.
- In November 2001, probably a hydrogen gas and an oxygen gas were locally accumulated and retained in a branch steam pipe elbow of the high-pressure injection system of the emergency core cooling system in Unit 1 of the Hamaoka Nuclear Power Plant of Chubu Electric Power Company. The gases were combusted or exploded for some reason to increase the pressure, which made the piping unable to withstand the pressure, resulting in ductile fracture.

(2) Fatigue Failure

The design codes and standards require that the postulated number of cycles of cyclic loadings to be postulated in the design should not exceed the allowable limit derived from the design fatigue diagram which is developed from data of fatigue fracture testing using a smooth specimen, by taking into account a design factor. The design fatigue diagram is developed so that no fatigue cracking occurs below the allowable limit. The codes also require that decrease in fatigue strength due to stress concentration can be considered for structural discontinuities. However, fatigue cracking may occur due to repeated loading on a high-stress area induced by causes such as the following.

- Occurrence of a weld defect in a weld described later caused by inappropriate workmanship
- Introduction of a structural discontinuity not postulated in the design that is caused by manufacturing error other than the above
- Application of large repeated loading not postulated in the design, and excess of the number of loading cycles.

With the same defect configuration and size, the stress intensity factor is larger if the defect occurs on the surface. Therefore, fatigue cracking tends to develop from defects on the surface.

The majority of failure in components that has actually occurred is fatigue fracture, which is said to represent more than 80%. Therefore, at the examination stage also, it is preferable to make a maximum effort to detect defects that will cause fatigue fracture. In particular, it is extremely important to detect surface defects.

The following are examples of accidents caused by this mode of failure at nuclear power plants.

- In February 1991, hydroelastic vibration occurred in an unsupported heat transfer tube of a steam generator in Unit 2 of the Mihama Power Plant of Kansai Electric Power Company because anti-vibration bar of the steam generator had not been inserted to the design depth. This caused a kind of fatigue failure called fretting fatigue in which relative sliding between a metal surface and the object in contact causes microcracks, part of which developed into penetration. As a result, leakage from the secondary system to the primary system occurred. Furthermore, the cracks extended to the rupture of the heat transfer tube, ultimately causing the emergency core cooling system (ECCS) to start up automatically. Following this accident, the then “Technical Standards on Structure etc., of Nuclear Power Generation Equipment” (announcement No. 501 by the Ministry of International Trade and Industry) was revised to add provisions concerning the control of the occurrence of hydroelastic vibration and the limitation of vibration amplitude.
- In December 1995, hydroelastic vibration caused by interaction between a sodium flow and a thermometer casing inserted in the secondary sodium system piping occurred on a thermometer installed in the secondary sodium system piping in

the prototype fast breeder reactor “Monju” of the Japan Atomic Energy Agency. In addition, since the root of the casing had been designed to be subject to stress concentration, fatigue fracture occurred in the stress-concentrated region. Finally, sodium flowed out through this fractured part.

- In July 1999, a hot bypass flow and a cold main flow were intermittently mixed near the outlet of the chemical and volume control system’s regenerative heat exchanger at Unit 2 of the Tsuruga Power Station of Japan Atomic Power Company because an inner duct was installed in the heat exchanger, and there was a bypass flow between the inner duct and the shell. Due to thermal fluctuations caused by this, cracking occurred in the elbows, etc., of the connecting piping of the heat exchanger, resulting in penetration in one of the elbows.

(3) Creep Fracture

As with ductile fracture, a component may be fractured if lack of the load-bearing capacity in terms of creep rupture strength in the creep temperature range to meet the design requirements is caused by fabrication errors and other errors. Creep voids or similar occur first, then they coalesce with each other, resulting in fracture.

(4) Failure Due to Creep-Fatigue Interaction

In the creep temperature range, the growth of fatigue cracks is accelerated by creep voids and others. Therefore, components fail in a shorter lifetime than the cases in which cracks propagate only due to fatigue. As with fatigue fracture, a high-stress region occurs due to introduction of a structural discontinuity or overload not postulated in the design, resulting in fracture due to repeated loading or similar in that region.

(5) Brittle Fracture

If the following conditions simultaneously occur, the probability of brittle fracture increases [5].

- If flaws or defects are developed at a stage such as material production, forming and component manufacturing, or fatigue cracks or others are initiated due to repeated loadings during service, these defects serve as a starting point of fracture.
- The material is embrittled due to decreased toughness resulting from welding or similar, unexpected progress of material deterioration as a result of aging, and improper use at low temperatures where toughness significantly decreases.
- A high-stress region occurs due to the presence of a structural discontinuity, weld residual stress, or overloading.

Thus, fracture frequently starts to develop from a defect developed in a weld. Therefore, the following section focuses on weld defects, specifies what configurations of defects occur and where such defects occur, and describes what kind of examination method is needed to detect the defects.

9.2.2 *Weld Defects*

This section first states measures to assure the quality of welds, including prevention of weld defects, and then describes the classification of weld defects.

[1] **Weld Defects and Quality Assurance**

According to the definition by the Japanese Industrial Standards, welding means an “operation of integrating two or more base materials using heat, pressure or both in order to provide continuity between the said base materials” [6]. This subsection discusses cases where base metals to be bonded are integrated through melting, as frequently practiced for nuclear power plants. Base materials are produced and formed through heat treatment and others under sufficient quality control at a factory of the respective material supplier and others. These base materials are welded by the component manufacturer at its factory or at the site of plant construction. Partly because it is not as easy to control the welding conditions as the conditions for base material production, the welded part (weld) can be subject to deterioration of its properties such as strength, ductility, and toughness due to flaws, alteration to a material different from the base material, or occurrence of residual stress or deformation that affect quality. Therefore, it is extremely important to appropriately specify a welding method and welding conditions and to have a welder with appropriate skills perform the welding according to the specified method and conditions, in order to assure the quality of the weld. In addition, it is essential to check the strength, ductility, toughness, etc., as well as to check for any weld defects after the welding.

Concerning weld defects, strenuous efforts have been made to appropriately detect weld defects after welding using the nondestructive examination methods described later. By contrast, the strength, ductility, etc., of a weld cannot be identified unless it is broken after the welding. Thus, those processes with which it is difficult to assure quality are regarded as special processes and require particular attention in the field of quality assurance. In the case of welding, which is a special process, it is required to demonstrate that the strength, ductility, etc., are appropriately ensured for the welding work to be performed, through a welding procedure qualification testing and others conducted in advance. Therefore, after the welding, examination must be performed for any weld defects. Configurations and locations of weld defects may vary according to various conditions including welding methods and types of base materials. For this reason, it is necessary to select a method for examining weld defects while fully understanding the characteristics of the potential defects.

The so-called welding standards define provisions such as what requirements welding methods should meet to assure the quality of welds, what requirements welders should meet to prove the appropriate skills, and what kinds of examinations should be performed to remove weld defects. The welding standards establish the following technical goals for welds of pressure retaining parts [7] of vessels and piping, as mentioned in Sect. 8.5.1, Chap. 8.

- (1) The configuration of the weld shall not be discontinuous or singular.
- (2) It shall be confirmed through nondestructive testing that the weld will not result in cracks and does not have incomplete penetration or any other defects that may be detrimental to ensuring sound welds.
- (3) The weld shall have an appropriate strength.
- (4) The welding shall be performed using a welding procedure that has been confirmed to be appropriate in advance through mechanical testing and others.

The above-mentioned technical goals or performance requirements are regulated also in the new examination/inspection system that was started in 2020. The item (2) above specifies the technical goal for weld defects regarding the nondestructive examination methods. It requires to remove all defects detected. It should be noted here that any examination method has the lower detection limit. The item (2) above requires substantially that any flaw below the detection limit should not be detrimental. This means implicitly to require sufficient fracture toughness not to cause unstable fracture in materials containing only flaws below the detection limit. The nondestructive examination method should be selected taking into consideration the allowable size of flaw from the aspect of detection limit.

[2] Classification of Weld Defects

A weld defect may occur even in weld zones which in base metal are not heated to the melting temperature but thermally affected, in addition to the areas where the filler metal inserted to the weld is melted and the base metal is melted. The degree of material alteration varies depending on the history of thermal impact on the relevant area, such as whether melting occurred or not. The form and occurrence probability of a flaw or defect also varies. Figure 9.1 shows the structure of a weld. The details of the weld are defined below [6].

- (1) Filler metal: Metal or alloy to be added during welding as what is called a filler rod and others. For example, in the TIG welding method, tungsten is used as an electrode to generate an arc between the electrode and the base metal (precisely speaking, weld pool), a filler rod is inserted into the arc and is melted, and also the base metal is melted.
- (2) Deposited metal: Metal that has drawn from the filler metal to the weld.
- (3) Fusion zone: Part of a weld where the base metal is melted.
- (4) Weld metal: Metal that is melted during welding and solidified, as a part of weld. It consists of deposited metal and a fusion zone.
- (5) Weld interface: Interface between the weld metal and the portion of the base metal which has not been melted.
- (6) Heat-affected zone (sometimes abbreviated as HAZ): The portion of the base metal which has not been melted, but whose microstructures, metallurgical properties, mechanical properties, etc., have been altered by the welding heat.
- (7) Weld: The portion including the weld metal and the heat-affected zone.

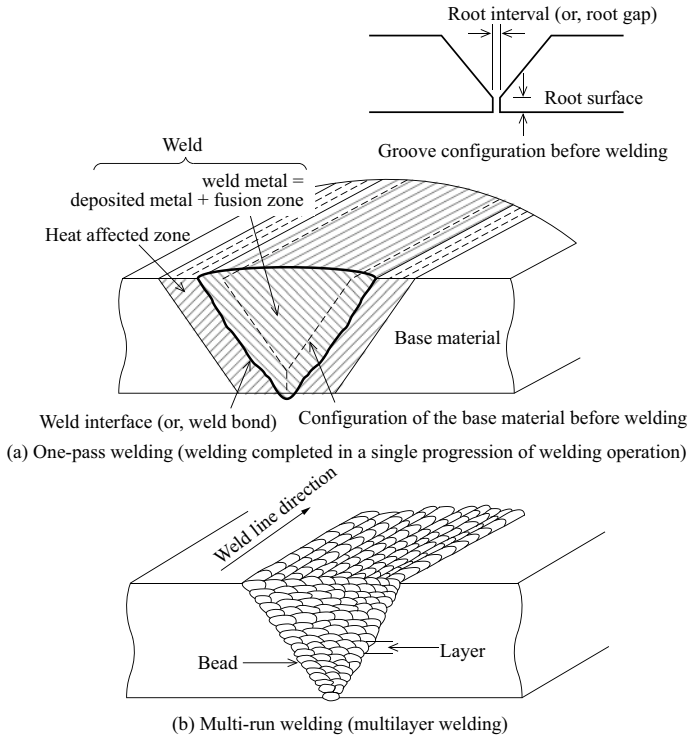


Fig. 9.1 Terms of weld

- (8) Welded joint: The junction of members or the edges of members which have been joined by welding.
- (9) Bead: A weld metal resulted from a pass. “Pass” means a single progression of a welding operation performed along the weld line. Figure 9.1a shows a weld completed in a pass. This is called one-pass welding. By contrast, Fig. 9.1b shows a weld in which the whole weld metal is formed in two or more passes, and this is called multi-run welding (or multiple pass welding). If a weld is produced in two or more layers, it is also called multilayer welding.

As partly described in Sect. 8.5.2 of Chap. 8, imperfections in welds, especially, weld defects can be categorized mainly by configurations as follows.

- (a) Crack: Cracking is classified into hot cracking (solidification cracking), cold cracking, and reheat cracking according to weld temperatures at which a defect occurs. It is also classified by locations into bead cracking, root cracking, lamellar tearing, etc.
- (b) Cavities (porosity): Cavities include spherical blow holes, cylindrical cavities (worm holes), and elongated cavities. Openings that reach the surface are called pits.

- (c) Inclusions: Solid foreign material mainly caught in the weld metal. For example, slag caught in the weld metal is called slag inclusions.
- (d) Lack of fusion/incomplete penetration: State in which any portion designed to melt and fuse to the base metal or the previous weld bead is not correctly fused to that, and a gap or other is left in the weld.
- (e) Incomplete configurations: Defective configurations are different from the designed weld configuration. They include undercut, excessive convexity, excessive root penetration, overlapping, linear misalignment, and angular misalignment (or angular distortion).
- (f) Others.

Figure 9.2 schematically shows these weld defects with their respective locations and characteristic features. Table 9.1 shows the definitions, causes, and locations of the above defects. This table also provides precautions on examination (issues in applying a specific examination method, in particular) and the impact of the defects on the strength of the weld or welded joint.

These descriptions can be summarized as follows.

- (1) Weld defects can occur throughout the weld, not only within the weld metal but also in the heat-affected zone. In addition, even a defect occurring within the weld metal may grow to reach the surface to form an opening in some cases. However, some types of defects occur only in limited places. For example, blow holes and slag inclusions occur only within the weld metal or the weld interface in principle.

Fig. 9.2 Examples of weld defects

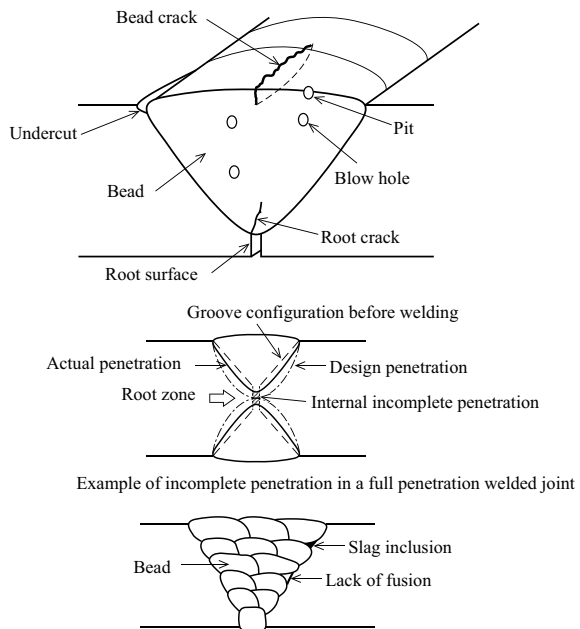


Table 9.1 Causes and locations of weld defects and issues in nondestructive examination

Weld defect	Definition (JIS Z 3001-4) ³⁾	Cause ⁷⁾
Crack	Hot crack Cracking that occurs at high temperatures within or immediately below the solidifying temperature range of the weld	Caused by tensile strains due to autogenous shrinkage (or solidification shrinkage), external deformation, etc. of the weld at high temperature during or immediately after welding. It is classified into cracking occurring at solidus temperature T_s or above, and cracking occurring below T_s . At or above T_s , cracking is caused by the behavior of the liquid remaining on the grain boundary around T_s (subdivided into solidification cracking and liquefaction cracking). Below T_s , cracking is caused by the behavior of the solid phase composition segregation on the grain boundary, deposit and dislocation (ductility deterioration crack).
	Cold crack Generic term for cracking that occurs when or after the weld temperature decreases to around normal temperature after the welding. This type of cracking includes root cracking, under-bead cracking and weld toe cracking.	It occurs after the weld of steel is cooled to below approx. 200°C or around normal temperature, and propagates with time. It is regarded as a type of delayed cracking in steels occurring due mainly to the stress and diffusible hydrogen, and the stress, hydrogen and metal microstructure in the crack point are involved in the cracking. Other than delayed cracking, this type of cracking also includes lamellar tearing, which is caused by laminated nonmetallic inclusion, distortion cracking, which is mainly caused by distortion, and quenching cracking, whose main cause is martensitic transformation.
	Reheat crack Cracking that occurs during reheating after welding	This cracking occurs during the post-weld heat treatment process or in service at high temperatures. It occurs through accumulation of distortion in the stress concentration zone as a result of stress relaxation when a welded joint under residual stress is heated to high temperatures at which creep deformation occurs. Intergranular cracking occurs as the transgranular structure is strengthened by fine carbide, as a precipitation strengthening element, or the grain boundary is embrittled through segregation of impurities.
Cavities	Also called porosity, they occur through inclusion of gases.	Blow holes, which are spherical cavities formed in weld metal, occur as follows. When the weld metal is overheated to sufficiently dissolve a gas into it, it releases the gas as the temperature decreases. When the weld is solidified, a large amount of gas is rapidly released to the solidification interface. Three main sources of the gas are assumed as follows. (1) Release of gas according to a change in solubility at or above the solidification temperature or the difference in solubility between the liquid and solid at the solidification temperature. (2) Production of gas through chemical reaction in the weld metal. (3) Physical inclusion of gas in the arc atmosphere. Thus, the gas dissolved in the weld metal is released, and when the gas pressure has reached or exceeded a certain value, bubbles are formed. If the solidification speed of the weld metal is larger than the traveling (floating) speed, these bubbles are entrapped in the solidified metal to form blow holes.
Inclusions	Foreign materials included in weld metal	By the names of foreign materials, they include slag inclusions and flux inclusions. Slag inclusions are slag entrapped in the weld metal without floating to the surface due to causes such as poor skills of the welder. Ungasified material from the coating that covers the welding rod becomes slag to refine and protect the weld metal and remove impurities.
Lack of fusion	State in which weld interfaces are not sufficiently fused into each other	In such cases as insufficient heat input due to too low welding current, resulting in failure to ensure penetration, the weld interfaces may not be fused into each other between the weld bead and the groove face or between beads.
Incomplete penetration	State in which actual penetration is not as sufficient as designed.	State in which the root face are not fused for causes such as failure to ensure sufficient welding heat input. In the case of X groove or similar, the root zone is inside the weld metal (this type of incomplete penetration is called internal incomplete penetration). In the case of V groove or similar, the root zone is on the surface of one side (called one-sided incomplete penetration).

(continued)

Table 9.1 (continued)

Weld defect	Location ⁷⁾	Issue in nondestructive examination ⁷⁾ , etc.
Crack	Hot crack	Issues in examination: particular attention is required in the case of radiographic examination. Cracks are a kind of planar (two-dimensional) defects, and the larger the angle between the defect surface and the irradiation direction of the radiation, the lower the detectability. Strength degradation: most likely to cause strength degradation among various kinds of defects.
	Cold crack	
	Reheat crack	
Cavities	Spherical blow holes, cylindrical worm holes, etc. are generated in the weld metal. A gas cavity formed when one of these bubbles reaches the surface of the weld to make an opening is called a pit (or surface pore).	Issues in examination: blow holes are spherical defects. In the case of ultrasonic examination, these holes do not have surfaces large enough to reflect ultrasonic waves, and therefore a large echo cannot be expected. Strength degradation: although they are not as detrimental as cracks, cavities with size several millimeters or larger could decrease the strength, lowering the fatigue strength in particular.
Inclusions	Observed in weld metal.	Issues in examination: slag inclusions are regarded as spherical defects. In the case of ultrasonic examination, these holes do not have surfaces large enough to reflect ultrasonic beams, and therefore a large echo cannot be expected. Strength degradation: they could decrease the strength of the joint.
Lack of fusion	Occurs in the bond zone and the weld metal.	Issues in examination: in the case of ultrasonic examination, it is desirable to take into account the incident direction of the ultrasonic wave regarding lack of fusion on the groove face and others. Strength degradation: it could decrease the strength of the joint.
Incomplete penetration	Depending on the location of the root zone etc., it occurs inside the plate or on the surface.	Issues in examination: in the case of ultrasonic examination, it is necessary to detect inner incomplete penetration, using the tandem detection method. Strength degradation: it could cause a structural discontinuity not assumed in the design, and decrease the strength of the joint.

- (2) Configurations of defects vary. They include spherical configurations such as blow holes, and planar configurations such as cracks. In addition, some defects, such as root crack, occur in structural discontinuities including root zones, where stress concentration occurs. Nondestructive examination of structural discontinuities is not easy, applicable examination methods are limited, and the examination requires proficient skills.
- (3) Defects of a certain size or greater cause decreases in the strength of welded joints.
- (4) Types of weld defects may also vary with types of base materials and welding methods. For example, hot crack is said to occur in stainless steel, heat-resistant alloys, or non-ferrous metals in many cases. Cold crack occurs in low/medium alloy high tensile strength steel and medium/high carbon steel in many cases. However, hot crack occurs also in low-carbon steel and other materials. Therefore, you should not judge whether the observed crack is of hot or cold type, based only on the material [8].
- (5) In terms of welding methods, most cracks resulting from electroslag welding are hot cracks.
- (6) According to the result of an actual defect survey (report of the 5th subcommittee for joint research in 1984 published by the Japan Welding Engineering Society [9]), in case of MIG (metal inert gas) and MAG (metal active gas) welding, blow holes account for approximately 1/3 of weld defects, lack of fusion slightly below 20%, followed by incomplete penetration, undercut, and others. Cracks, which are detrimental, are reduced to represent a relatively small proportion at 3 to 5%, as a result of efforts to prevent their occurrence. Another survey indicates that types of common defects vary also with weld configurations, such as butt welding and fillet welding, and with industries. For example, in the shipbuilding industry, incomplete penetration represents an overwhelming majority of weld defects in butt welding [10].

Meanwhile, nondestructive examination methods for detecting these defects have different advantages and disadvantages. They are briefly mentioned in Table 9.1. Details are described in the next section. Therefore, it is preferable for the examiner to first fully grasp that the type and property of the weld defect varies with the welding method and welding conditions, select an appropriate examination method while considering the performance and quality level required for the welded joint, and then appropriately set criteria on whether or not the flaw is a defect.

The recent trend concerning the relationship between welding methods and defects is as follows. If the welding methods and welding conditions are strictly limited, and a qualified welder or an automatic welding machine having sufficient reliability is utilized, then it is possible to achieve defect-free. On the other hand, nondestructive examination of a weld cannot be performed because of a limited accessibility in some cases and others. Therefore, if it is possible to certainly assure that no significant defect will occur as required under the design code, etc., within the limited range of fabrication conditions, it may be considered unnecessary to perform defect

examination (it is proposed that this concept be called defect-free or examination-free fabrication technology [11]). Such a case has not been reported concerning light-water reactor power plants. However, the adoption of this concept for vacuum vessels and superconductive coils of tokamak fusion systems is under consideration.

9.3 Nondestructive Examination

9.3.1 Categorization

Before describing the classification of nondestructive examination methods, it is better to establish a distinction between the meanings of “nondestructive examination” and “nondestructive testing” here. As defined in Sect. 9.1, “nondestructive testing” can be simply rephrased as testing to identify whether a particular structure has a flaw or defect. On the other hand, “nondestructive examination” (or nondestructive inspection) includes evaluation and judgment of whether the structure meets the acceptance criterion (i.e., whether the detected imperfection is a flaw, which is not necessarily rejectable, or detrimental defect) based on the result of the testing.

The purpose of nondestructive testing is to check for any flaw or defect on the surface and within the body of a structure and know its state without destroying it, as a means of quality control of structures. Nondestructive testing always uses some kinds of physical phenomenon and is used to detect a flaw or defect, and in some cases, to additionally identify its configuration, size, etc., in other words, to perform measurement (sizing). Focus is placed on detection in the construction stage because all detected defects are required to remove. On the other hand, in the operation stage, it is placed on sizing in order to measure the size, etc., because the defect should be evaluated whether during operation it grows to such a critical size as to cause unstable fracture. Further, even if some nondestructive testing methods are based on the same physical phenomenon or principle, their performances such as detectability and measurability are not the same as each other; for example, the method A has an excellent detectability but less measurability, whereas the method B has an excellent detectability and good measurability. We need to carefully choose the nondestructive method to meet the requirements such as the testing purpose, defect configuration to be detected and measured (for instance, spherical defect vs. planar defect), and accuracy. Major nondestructive testing methods can be categorized as follows from the viewpoint of employed physical phenomena or principles [12].

- (1) Testing methods employing principles of optics or chromatics: appearance testing (or visual testing), liquid penetrant testing, magnetic particle testing.
- (2) Testing methods employing radiation principle: radiographic testing.
- (3) Testing methods employing electromagnetic principle: magnetic particle testing, eddy current testing.
- (4) Testing methods employing acoustics principle: ultrasonic testing, acoustic emission (AE) testing.

Acoustic emission testing is a method of measuring the sound (elastic wave) that is emitted in a structure when it is deformed or fractured.

- (5) Testing methods employing thermology principle: infrared thermography testing.

Infrared thermography testing is a method of photographing the temperature difference on the surface using an infrared camera. This method uses the principle that if heat is externally applied to an object containing an internal defect or similar, there will be a difference in surface temperature between defective and sound zones.

- (6) Testing methods employing leakage principle: leak testing.

In addition to the above, methods using new principles and those derived from upgrading of conventional methods have recently been proposed, and research and development aimed at putting those new methods into practical application are being conducted. Methods using new principles include, for example, the electric potential method, which was previously used as a method of fracture mechanics testing. The upgrading of conventional methods will be mentioned in the sections describing respective methods.

These testing methods can be roughly divided into two groups; one group includes the methods that physically affect the surface of the object and enable you to acquire only information on the surface, such as liquid penetrant testing, and another includes the methods that also affect the interior of the object and enable you to acquire information on the interior. Those affecting the interior also affect the object from outside; in some cases, their effects may be limited to the surface layer, and in the other cases, those reach the deep part of the interior and you can acquire information on the backside. In the following sections, those testing methods enabling you to acquire information on the surface layer are discussed in the same category as the testing methods providing information on the surface. Thus, testing methods are classified as follows, according to the part on which information can be acquired.

- (a) Nondestructive testing to acquire information on the surface or the surface layer (generally called “surface testing”): appearance testing, liquid penetrant testing, magnetic particle testing, etc.
- (b) Testing to acquire information on the interior (generally called “volumetric testing”): radiographic testing, ultrasonic testing, etc.

Concerning the surface layer, if the wall of piping is thin, as is the case with steam generator heat transfer tubes of a PWR, the whole pipe wall may be dealt with as a surface layer. In such a case, the testing may be classified as (b). Therefore, you should take care not to be confused in the classification.

Table 9.2 shows the principles of flaw/defect detection, defect locations, and detectable flaws and defects in welds for representative nondestructive testing methods in a comparative manner.

Table 9.2 Nondestructive testing methods for detecting flaws and defects [12, 14]

Testing method	Volumetric testing		Surface testing		
	Radiographic testing (RT)	Ultrasonic testing (UT)	Magnetic particle testing (MT)	Liquid penetrant testing (PT)	Eddy current testing (ECT)
Detection principle	X-ray and γ-ray permeability Detect a flaw/defect based on the difference in transmitted dose between sound and flaw/defect sections.	Reflection of ultrasonic pulse Detect a flaw/defect by receiving an ultrasonic echo reflected from the flaw/defect.	Magnetic attraction Magnetic leakage flux is generated at a flaw/defect, and magnetic particles adhere to the flaw/defect.	Capillary action of penetrant Detect a flaw/defect by applying a penetrant to the surface, then causing it penetrating the flaw/defect and using a developer to cause the penetrant to be drawn out of the flaw/defect (blotting effect).	Electromagnetic induction A flaw or defect causes a change in the eddy current induced by alternating magnetic flux. This produces a change in the output of the detection coil and allows detection of the flaw/defect.
Location of the defect	Inside the test object	Mainly inside the test object	Surface and surface layer of the test object	Surface of the test object	Surface and surface layer of the test object
Detectable flaws/defects in the weld	Blow hole, incomplete penetration, lack of fusion, slag inclusion, cracks	Cracks, incomplete penetration, slag inclusion, concentrated blow holes	Linear defects such as cracks in a ferromagnetic body	Cracks, pit, etc. open on the surface	Cracks, etc. in a conductor. Although welds were previously difficult to detect because of lift-off noise, the detectability has been improved recently.

9.3.2 Characteristics of Nondestructive Testing Methods

This section describes representative test methods with focus mainly on radiographic testing because of its widely spread use.

[1] Radiographic Testing (RT)

Among radiation rays, X-ray and γ -ray are electromagnetic waves with very short wavelengths, and are highly capable of transmitting through substances. As it is widely known, Roentgen discovered that the interior of substances can be observed using X-ray in 1895. In 1896, X-ray examination was performed to identify the position of a bullet in a human head. There is no essential difference between X-ray

and γ -ray. While X-ray is generated outside an atomic nucleus, γ -ray is generated from an atomic nucleus. If X-ray or γ -ray is radiated to a target structure (e.g., a plate) and a radiographic film is placed behind the opposition side of the structure, the film will be exposed to the X-ray or γ -ray that has transmitted through the structure. Since the intensity of transmitting X-ray differs between flaw and sound sections, the degree of exposure differs, which is shown on the film as different shades. For example, if a weld has a cavity such as a blow hole and crack, or a foreign material such as a slag inclusion, the X-ray transmitting through the cavity does not attenuate as much as that transmitting through a sound section, and the respective part of the film is exposed more intensely. As a result, only the part of the developed film corresponding to the presence of a defect will look dark [13]. Radiographic testing detects flaws and defects within a structure by using this principle.

For structures with thick walls, high-energy X-ray or γ -ray is used. For thin-wall structures, low-energy X-ray is used.

Through defect detection using radiographic testing, the location, distribution, configuration, and dimensions of a defect can be specified as objective information from the produced radiographs, and it is easy to distinguish between defect types [14]. In addition, this method provides good repeatability for important testing results as a means of defect testing used in quality control and excels in recordability since testing results are preserved in the form of radiographs [14]. Owing to these advantages, radiographic testing is recognized as a means to ensure quality assurance of welded joints, and thus assure structural integrity, and widely adopted as a defect detection method under the structural code.

(1) Principle [12, 14]

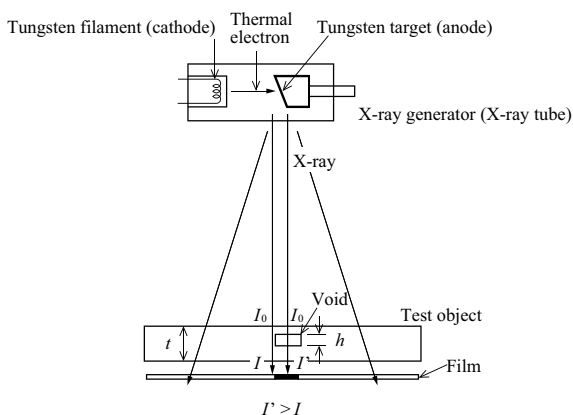
An X-ray generator consists of an X-ray tube and other components. In the X-ray tube, if a high voltage is applied to the two poles, which are a filament (tungsten cathode) and a target (tungsten anode) as shown in Fig. 9.3, thermal electrons emitted from the filament are accelerated and collide with the target. The majority of the kinetic energy of electrons is transformed into heat, and part of the energy is radiated as X-ray. This X-ray enters the object to be tested (test object) in the radiation field of a certain size. Now, take a case in which a defect with height h , a void for example, is present in the test object with thickness t as shown in Fig. 9.3. When the X-ray transmits through the test object, it causes an interaction such as photoelectric effect with the substance constituting the test object, and then attenuates. The intensity I of direct X-ray that transmits through a sound section having no defects and reaches the X-ray film can be expressed by the following equation.

$$I = I_0 e^{-\mu t} \quad (9.1)$$

where I_0 represents the intensity of radiation entering the test object, and μ is an absorption coefficient (or attenuation coefficient).

Concerning the X-ray transmitting through the void with height h , its penetration thickness is given by $t - h$, and the intensity I' of the directly penetrating radiation

Fig. 9.3 Concept of radiographic testing (in the case of X-ray)



can be expressed by the following equation.

$$I' = I_0 e^{-\mu(t-h)} = I + \Delta I \quad (9.2)$$

Therefore, on the part of the film exposed to the X-ray transmitted through a defect, the intensity of transmitted radiation is greater by $I' - I = \Delta I$ compared with the part exposed to the X-ray transmitted through a sound section. As a result, a difference in blackening, or density difference, proportional to the difference in X-ray intensity ΔI is produced on the X-ray film, allowing the defect to be detected as an image.

(2) Detectable Defects [12]

Even small flaws and defects can easily be detected if there is a difference in thickness in the transmitting direction: e.g., a blow hole, slag inclusions, and other inclusion in a weld. However, if the thickness is small even if the defect has a relatively large area as is the case with a crack, the difference in the thickness of the material through which X-ray transmits is very small when the angle of radiation relative to the crack surface is large, and therefore it is difficult to detect the defect.

The same applies to a lack of fusion, which may not be detected unless X-ray enters the material along the surface of the lack of fusion.

(3) Precautions [14]

Basic precautions (including the procedure for classifying defects) on performing radiographic testing are addressed in JIS Z 3104 standard “Methods of radiographic examination for welded joints in steel” [15]. However, this standard only defines the minimum requirements. To determine specific testing conditions and ensure reliable film development, it is essential that the engineers involved are well qualified. In addition, to perform acceptance/rejection judgment of tested sections based on

the result of radiographic testing, the examiner must have knowledge on materials, welding, and design specifications and service conditions of the test object, etc., in addition to radiographic testing techniques, and a certified engineer with sufficient skills must be appointed for this duty.

Since this testing involves radiation, it is necessary to designate the test site as a controlled area before conducting the testing. It is necessary to observe the following laws and regulations, strive to minimize the radiation exposure of not only the personnel involved in the radiation work but also those in the vicinity as low as reasonably achievable, and prepare the required facilities and establish a management structure.

- “Act on the Regulation of Radioisotopes, etc.” (formerly, Act on the Prevention from Radiation Hazards due to Radioisotopes and others” (so-called “Radioisotopes Regulation Act” or in abbreviation, “RI Act”)
- “Rules for Prevention from Hazards from Ionizing Radiation” (so-called the Ionization Radiation Rules) in particular among the Industrial Safety and Health Act and Related Ordinances.

When handling radiographs of welds of austenitic stainless steel, you must pay a special attention. Even though a flaw or defect is not present actually, images called “spurious (false) flaw images,” whose configurations look like flaws or defects, frequently appear in such radiographs [16]. It has been proved that the cause of this is a diffraction phenomenon of X-ray due to the solidified microstructure of the weld. JIS Z3106 Methods of radiographic examination for welded joints in stainless steel Annex 5 (reference) [17] provides guidelines on the method for distinguishing between “spurious flaw images” and actual flaw images. By referring to the guidelines, it is necessary to take care not to confuse those images with images of flaws in the weld such as weld defects and blow holes.

(4) Recent Trend

Various elementary technologies have been sophisticated. For example, concerning X-ray generators, development of small linac X-ray sources is in progress with the aim of downsizing generators to make them portable, and eliminating the need to establish a controlled area [18]. In addition, an X-ray image examination system based on a “reusable film” using an imaging plate has recently been developed. This system is much more sensitive than the conventional films and has many advantages such as the ability to digitally process acquired images. The rules for the above-mentioned examination method are provided in the ASME Boiler and Pressure Vessel Code Section V Nondestructive Examination (NDE). In Japan, similarly, the Japanese Industrial Standard JIS Z 3110: 2017 Non-destructive testing of welded joints—Methods of radiographic testing for X- and gamma-ray techniques with digital detectors was issued in 2017, referring to ISO 17636-2:2013, Non-destructive testing—Part 2: X- and gamma-ray techniques with digital detectors.

[2] Ultrasonic Testing (UT)

Unlike sound waves audible to human ears (audible sound), an ultrasonic wave, when propagates through a metallic test object, transmits to a limited range like a light from a lighthouse. In other words, it is highly directional and travels straight in a constant direction in the form of a clean-cut bunch of sound. An ultrasonic wave traveling straight has the principle of being reflected on the interface between different objects or between a void and a material. Ultrasonic testing detects a flaw or defect in the test object and measures its position and size by using such a principle.

(1) Principle [12, 14]

As shown in Fig. 9.4, generally a voltage is applied across a transducer (oscillator) consisting of a piezoelectric material to whose both opposite faces electrodes are attached, and it produces mechanical vibrations. Or reversely, an electromotive force is generated if vibrations transmit to the transducer. A probe containing this transducer is placed on the surface (called scanning surface) of the test object. As the transducer oscillates in pulses, the particles consisting of the metallic test object as the aggregate are oscillated and the oscillation propagates in the test object as an ultrasonic pulse. If a void is present between the probe and the scanning surface when the transducer in the probe comes in contact with the scanning surface, the ultrasonic wave will be completely reflected on the interface with the test object due to a large difference in acoustic impedance between the void and the test object and will not enter the test object. Therefore, a liquid contact medium such as water and glycerin is applied to the test object to expel the air from the void. The ultrasonic wave transmitting the test object is reflected on any defect. When the reflected ultrasonic wave (this is called an echo) returns and transmits to the transducer, a voltage is produced between the electrodes.

A change in this voltage is transmitted to an ultrasonic detector as a signal and displayed on a screen of a display (e.g., cathode-ray tube, or CRT). The horizontal axis of the display is also called a time axis, and a value proportional to the propagation time between the transmission of the ultrasonic wave from the transducer and the reception of the returned ultrasonic wave is expressed along this axis. This propagation time is proportional to the distance that the ultrasonic wave traveled. The vertical axis indicates a signal, and the height of the indicated signal is proportional to the intensity of the received ultrasonic wave. The height of the indicated signal is called echo height. Therefore, the location of the flaw or defect is determined by the time until the transmitted ultrasonic wave is received. The size of the flaw or defect is estimated by the height of the received echo displayed on the display or on the range of the displayed echo.

An ultrasonic wave can propagate in the test object as a longitudinal wave and a transverse wave. It also contains a surface wave, which propagates on the surface, and a plate wave, which vibrates the entire thin test object. In a case shown in Fig. 9.4a, particles vibrate in the perpendicular direction to the scanning surface (i.e., the vibration direction of the piezoelectric material) and an ultrasonic wave propagates in the same direction. This wave is called a longitudinal wave (compressional wave). By

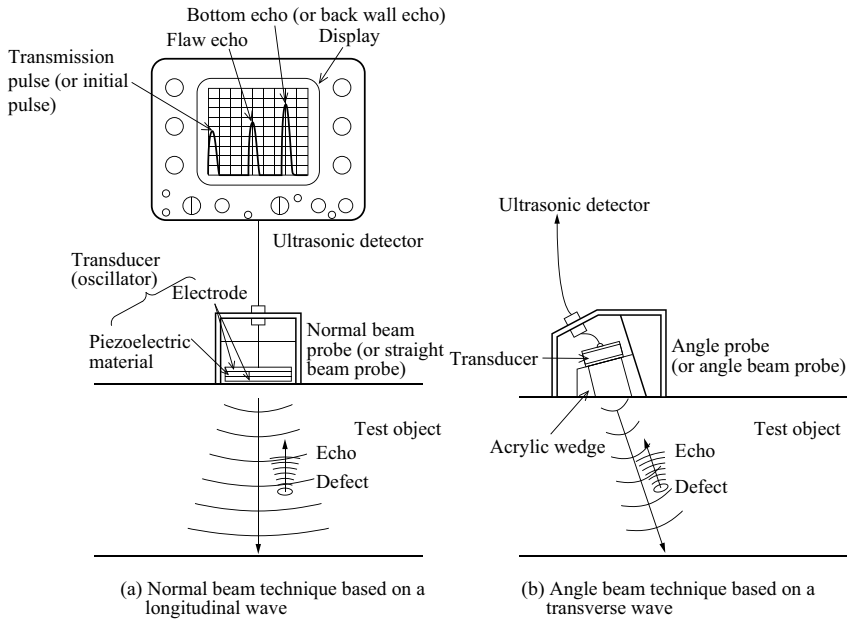


Fig. 9.4 Concept of ultrasonic testing

contrast, a wave with which particles oscillate at the right angle to the propagation direction of the ultrasonic wave is called a transverse wave (shear wave). To explain the conditions for a transverse wave to propagate in the test object, let us examine a case in which an ultrasonic wave enters the surface of the test object at an oblique angle. In this case, as shown in Fig. 9.4b, a transducer is mounted on an acrylic wedge, and an ultrasonic wave is generated to propagate in the acrylic and then propagate in the test object via the acrylic wedge. Although the ultrasonic wave propagates in the wedge as a longitudinal wave, it is refracted in the interface between the wedge and the test object because the sound speed differs between the wedge and the test object, and this is where a transverse wave is produced. Therefore, a longitudinal wave and a transverse wave may simultaneously propagate in the test object. This would make it difficult to analyze the defect. Therefore, it is necessary to adjust the incident angle of the ultrasonic wave into the test object by considering Snell's law, which expresses the relationship between the propagation speed of a wave propagating through two media and the incident and refracting angles, so that the longitudinal wave is refracted and only the transverse wave propagates in the test object.

There are two ultrasonic testing techniques as follows, distinguished by the incident angle of the ultrasonic wave to the scanning surface.

- Straight beam technique in which an ultrasonic wave perpendicularly enters the scanning surface and propagates: a normal beam probe, as shown in Fig. 9.4a, is used as the probe.

- Angle beam technique in which an ultrasonic wave enters the scanning surface at an oblique angle and propagates: an angle probe, as shown in Fig. 9.4b, is used as the probe.

(2) Detectable Defects

For ultrasonic testing, appropriate testing methods vary according to the purpose of detection (types of flaws or defects to be detected). Table 9.3 summarizes the testing methods and respective applications.

The size of defects detectable with ultrasonic testing is on the level of ultrasonic wavelength. In many cases, the frequencies of ultrasonic waves used for detecting defects of steel materials are 2 MHz or 5 MHz. In the case of 5 MHz, the sound speed in the steel material is 5900 m/s in a longitudinal wave and 3230 m/s in a transverse wave. Therefore, the corresponding wavelengths are 1.2 mm and 0.64 mm, respectively [14]. To detect a smaller defect, it is necessary to increase the frequency. For example, a ceramic tends to have a small fracture toughness value, and it is necessary to detect an initial defect (manufacturing defect) of 0.1 mm or less. Therefore, the examiner tends to apply an ultrasonic wave of tens of MHz. However, the higher the frequency, the more easily scatter reflection occurs and the larger the attenuation of the ultrasonic wave becomes.

(3) Precautions

Basic precautions on ultrasonic testing are described in the following Japanese Industrial Standards.

- JIS Z 2344: General rule of ultrasonic testing of metals by pulse echo technique
- JIS Z 3060: Methods for ultrasonic examination for welds of ferritic steel.

Austenitic stainless steel welds, weld metals in particular, are of large grain structure, and therefore, an ultrasonic wave is substantially attenuated and it is necessary to use an ultrasonic wave with a long wavelength for propagating a long distance. As a result, its resolution for flaws and defects is low. In addition, in the angle beam testing using a transverse wave, the straightness of an ultrasonic wave is deteriorated by sound velocity anisotropy, etc., and a spurious echo (forest-type echo) occurs [19]. As a result, a defect echo cannot be discriminated in some cases. To solve this

Table 9.3 Methods of ultrasonic testing and their major applications [12]

Testing method	Wave type	Major applications
Normal beam	Longitudinal wave	Examination of internal flaws/defects of cast materials, forged materials, rolled materials, etc., and thickness measurement
Angle beam	Transverse wave	Examination of internal flaws/defects of welds, pipe materials, etc.
Surface wave examination	Surface wave	Examination of surface flaws/defects
Plate wave examination	Plate wave	Examination of flaws/defects of thin plates

problem, many research activities have been conducted and guidelines proposed, including that a longitudinal wave angle beam probe should in principle be used for complete penetration butt welds with thickness 9 mm or more [20].

(4) **Recent Trends** [14]

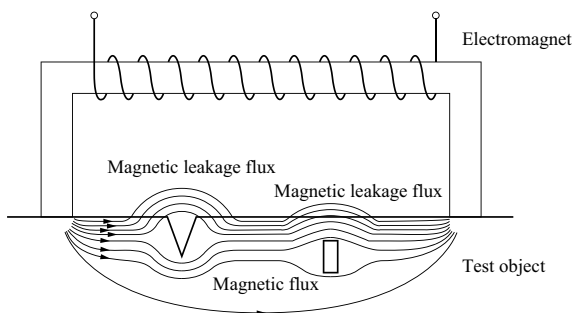
Ultrasonic testing techniques that have been developed include the Time-of-Flight Diffraction (TOFD) method, phased-array method (electronic scanning probe), and Electro-Magnetic Acoustic Transducer (EMAT). The TOFD method has been developed as a method that can compensate for the disadvantage of ordinary ultrasonic testing, which is inferior to radiographic testing in the ability to image and record testing results. The TOFD method is capable of clearly detecting not only surface defects but also blow holes, which ordinary ultrasonic test methods cannot easily detect. The EMAT method is a non-contact ultrasonic test method, using magnets and coils. The coil with high-frequency current induces a magnetic field. This induced AC magnetic field produces eddy current in paramagnetic materials or magnetostriction and others in ferromagnetic materials. The eddy current in the magnetic field generates Lorentz force. The periodic fluctuations in the Lorentz force and magnetostriction produce ultrasonic waves. Then, the ultrasonic waves propagate in the material. Oscillations that are caused by returned ultrasonic waves produce induced currents in the magnetic field. The induced current generates electromotive force in the coil that turns into an electrical signal as received.

[3] **Magnetic Particle Testing (MT)**

(1) **Principle** [12, 14]

If a test object made of a ferromagnetic material such as carbon steel is magnetized with a magnet (this is called the yoke method), or magnetized with a large electric current (this is called the prod method), a flow of magnetic flux is produced in the test object. If a flaw or defect is present in the path of the magnetic flux flow, the flow is obstructed and is diverted as shown in Fig. 9.5 since neither a flaw nor defect contains a ferromagnetic substance. If this flaw or defect is open on the surface or near the surface, part of the diverted magnetic flux is scattered beyond the surface of the test object and is leaked into the air. This is called a magnetic leakage flux. Let us assume a case in which a magnetic flux flows from left to right as shown in Fig. 9.5. An N pole is formed in the place where some of the magnetic flux leaves into the air, and an S pole in the place where the flux re-enters the test object. Therefore, a small magnet is formed around each flaw or defect. If fine magnetic particles (ferromagnetic particles, normally several to tens of μm) are scattered, the magnetic particles are magnetized to form a small magnet having magnetic poles at both ends, and the magnetic particles are connected with each other and concentrated and attracted to the flaw or defect to produce a pattern of magnetic particles (this is called a magnetic particle pattern). The width of this magnetic particle pattern is several times larger than the width of the actual flaw or defect. To easily identify the magnetic particle pattern formed on the defect, the magnetic particles are specially

Fig. 9.5 Concept of magnetic particle testing (leakage of magnetic flux)



treated in advance so that their color or brightness will look different from the defect surface. This is the principle of magnetic particle testing.

(2) Detectable Defects

Although this method is applicable only to ferromagnetic materials, it is capable of detecting a defect as long as its depth is approx. 2–3 mm or less from the surface, even if the defect is not open on the surface or filled with nonmetallic inclusions.

(3) Precautions

Basic precautions are described in the following Japanese Industrial Standards:

- JIS Z 2320 Non-destructive testing—magnetic particle testing.
 - JIS Z 2320-1: 2017 Part 1: General principles.
 - JIS Z 2320-2: 2017 Part 2: Detection media.
 - JIS Z 2320-3: 2017 Part 3: Equipment.
- JIS Z 2319 Methods for magnetic flux leakage testing.

[4] Liquid Penetrant Testing (PT)

(1) Principle [12, 14]

PT is a testing method that uses two physical phenomena (capillary and perceptual phenomena) to make a fine flaw or defect open on the surface of the test object easily visible to human eyes. If water is poured into a crack or pit that breaks the surface, it will rapidly penetrate the defect. This is caused by the capillary phenomenon. This is characterized by applying a liquid (penetrant) having a color or fluorescence easily perceptual to human eyes, instead of water, to the surface and letting the liquid penetrate into cracks and the like. This is also called color check.

At first, as a pretreatment, any contaminant such as rust on the surface of the test object should be removed, and any foreign material inside the defect is removed to open the defect. Then, the test object is dried. Subsequently, a penetrant is applied to the surface as shown in Fig. 9.6 (this is called application of a penetrant). The

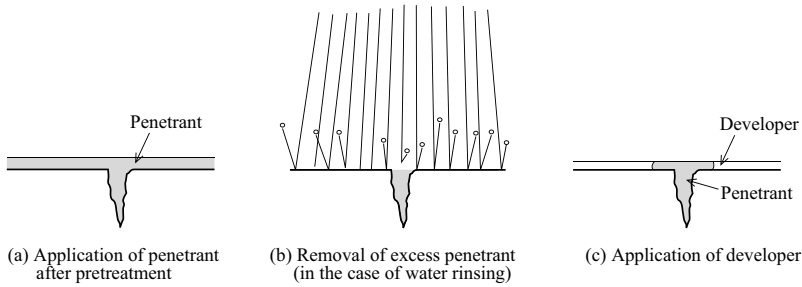


Fig. 9.6 Concept of liquid penetrant testing [14]

penetrant dwell time (or penetration time) is approximately five minutes to one hour. The penetrant must not be dried during the penetration time. A liquid penetrant method using a dyeing solution as the penetrant is called the dye penetrant method, while a method using a fluorescent agent is called the fluorescent penetration method. Then, the excess penetrant adhering to the object surface is rinsed and wiped off (removal of the excess penetrant). Naturally, the penetrant that has penetrated inside the cracks and the like must not be removed. Subsequently, a developer is thinly applied to the object surface (application of a developer). It enables the developer to draw and absorb the penetrant penetrating inside the cracks and the like. For example, if the penetrant is a red dye and the developer is white, a red pattern will surface. This pattern is called indication, which is to be observed.

When observing the pattern, the examiner must pay attention to any spurious pattern by using a white light for a dye penetrant and an ultraviolet ray for a fluorescent penetrant, in order to check for any defect and observe its properties.

(2) Detectable Defects [14]

For liquid penetrant testing, there are six types of penetrants including water-washable dye penetrants and solvent-removable dye penetrants, and four development methods including dry development, wet development, and solvent-based development. It is important to appropriately combine a penetrant and a development method to make the most of their characteristics according to the defect type and the detection strategy. In many cases, the combination of the solvent-removable dye penetrant testing and the solvent-based development method is adopted.

Although PT is to detect surface-breaking defects, it is difficult to detect flaws and defects without being hollow inside.

(3) Precautions

As mentioned earlier, characteristic defects occur depending on the welding methods in many cases. In addition, the testing result varies depending on the skills of the testing engineer. Therefore, it is important for the testing engineer to have sufficient knowledge on the expected defect types and testing methods.

Basic precautions are described in the following Japanese Industrial Standard:

- JIS Z 2343: Nondestructive testing – liquid penetrant testing.
 - JIS Z 2343-1 Part 1: General principles—method for liquid penetrant testing and classification of the penetrant indication.
 - JIS Z 2343-2 Part 2: Testing of penetrant materials.
 - JIS Z 2343-3 Part 3: Reference test blocks.
 - JIS Z 2343-4 Part 4: Equipment.
 - JIS Z 2343-5 Part 5: Penetrant testing temperature higher than 50 °C.
 - JIS Z 2343-6 Part 6: Penetrant testing temperature lower than 10 °C.

[5] Eddy Current Testing (ECT)

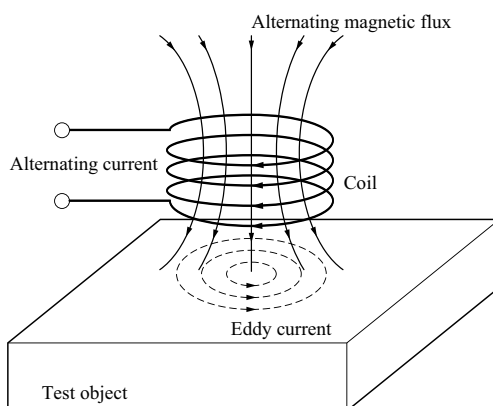
(1) Principle [14]

If an alternating current is applied to a coil, an alternating magnetic flux is produced. If this coil is brought in the proximity of a conductor, the magnetic flux develops in the conductor. Since the alternating magnetic flux changes with time, an electromotive force is generated as known as electromagnetic induction, and an electric current is induced in the conductor. Figure 9.7 schematically shows this. The current induced in the conductor through electromagnetic induction is called an eddy current.

The eddy current varies with not only the configuration and dimensions but also electromagnetic properties of the conductor and also depends on the presence or absence of discontinuity such as cracks. Therefore, it is possible to detect cracks and the like by detecting the eddy current. The nondestructive testing using this electromagnetic induction is called electromagnetic induction testing, and for the purpose of flaw detection, it may be called eddy current testing.

This testing method can be performed at high speed (several to tens of m/s) without contacting the test object. Although the method is capable of determining the size of the defect from the defect signal, it is difficult to discriminate defect types or quantitatively evaluate defects.

Fig. 9.7 Generation of eddy current by electromagnetic induction [14]



(2) Detectable Defects [14]

When an eddy current is induced in a conductor due to an alternating magnetic flux, the eddy current acts in a way to counter the magnetic flux, and therefore the magnetic flux attenuates rapidly in the conductor. As a result, the density of the eddy current decreases exponentially with the depth from the surface. This phenomenon is known as the skin effect. Therefore, almost no eddy current flows in the body of the test object. For this reason, eddy current testing is used for detecting defects on the surface or the surface layer in principle. The detectable depth is given by the following equation using the standard penetration depth δ [m] as an index.

$$\delta = \frac{1}{\sqrt{\pi f \mu \sigma}} \quad (9.3)$$

Here, f represents an alternate current frequency [Hz], μ magnetic permeability [H/m], σ electrical conductivity [S/m], and δ the below-surface distance at which the density of the eddy current is reduced to approximately 37% of the surface density. However, it should be noted that if this method is used for detecting defects in the steam generator heat transfer tubes of a pressurized water reactor (PWR), it is practically possible to examine through the tube wall thickness because the tube wall is thin.

(3) Precautions

Basic precautions are described in the following Japanese Industrial Standards:

- JIS Z 2316 Non-destructive testing—Eddy current testing.
 - JIS Z 2316-1 Part 1: General principles.
 - JIS Z 2316-2 Part 2: Instrument characteristics and verification.
 - JIS Z 2316-3 Part 3: Probe characteristics and verification.
 - JIS Z 2316-4 Part 4: System characteristics and verification.
- JIS G 0583 Automated eddy current examination of steel pipes and tubes.

(4) Recent Trend [14]

Since welds have excess weld metal in general, it is difficult to keep a constant gap between a typical coil (surface coil) and the test object, and therefore, a large noise (lift-off noise) has previously occurred due to a change in the relative position (lift-off) between the test object and the coil. Therefore, it was considered difficult to apply eddy current testing for defecting flaws in welds. However, an eddy current testing probe that in principle does not produce lift-off noise was developed recently, making it possible to apply eddy current testing for detecting flaws in welds.

9.4 Importance of Components and Nondestructive Examination

The impact of component fracture, including safety impact such as human damage and impact in view of property preservation, varies with the risks that the respective component has and the required functions. Meanwhile, the possible causes of fracture and failure, as described in Sect. 9.2, include the following:

- (1) Deformation develops due to the application of a load to initiate a void or similar, and it grows into a defect, resulting in fracture/failure.
- (2) A defect occurs during the fabrication, resulting in fracture/failure.
- (3) Higher stress than the level postulated in the design is generated due to lack of strength or design error.

In addition, it is also necessary to consider the likelihood of fracture and failure for the respective component. The severity of the applied load, or the generated stress in other words, varies depending on the component or the part. For example, in a case of a welded joint, as mentioned in the beginning of this chapter, the circumferential stress from the internal pressure is approximately twice the axial stress. Therefore, if the welded joint is formed in the axial direction of the vessel (this is called a longitudinal joint), the probability of failure is higher than the circumferential direction (this is called a circumferential joint).

Taking into account those mentioned above, to rationally assure the structural integrity of a component, it is necessary to work out specific measures by comprehensively considering various factors such as the risks that the component has, its importance, modes of potential failure, possible causes of the failure, and the likelihood of failure. Nondestructive examination methods have different advantages and disadvantages. Therefore, when considering how to combine specific nondestructive examination methods, it is of great importance to take these factors into account.

The existing structural codes and standards [21, 22] define the selection of combinations of nondestructive examination methods to be applied for the acceptance examination of materials and weld examination at welding, from the following two viewpoints.

(1) Importance of Components, etc.

As described in the concept in Sect. 5.5, regarding facilities for design basis events, the components, etc., of light water reactors are roughly classified in the order of importance as follows.

- (a) Class 1 components, etc. (vessels, piping, pumps, valves, and supports): those constituting the reactor coolant pressure boundary.
- (b) Class MC vessels, etc. (vessels and supports): those installed to prevent leakage of hazardous substances such as radioactive materials released from the reactor containment vessel, and from machinery or instruments connected to the containment vessel.

(c) Class 2 components, etc. (vessels, piping, pumps, valves, and supports):

- Those that may indirectly cause radiation hazards to the public through their trouble, failure, etc., belonging to equipment required to safely shut down the reactor or required to ensure safety in case of emergency.
- Those that, as one of the key functions, are related to circuits for circulating the fluid to drive turbines, ranging from the connections to Class 1 components to the nearest stop valves.
- Other than the above, those of the portions extending from a reactor containment penetration to the inner or outer isolation valve.

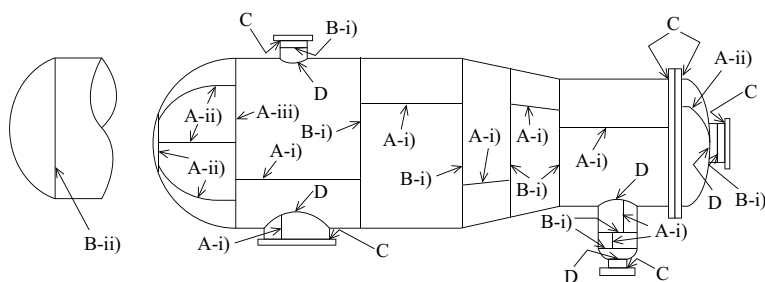
(d) Class 3 components, etc. (vessels, piping, pumps, valves and supports): components, etc., excluding Class 1 components, etc., Class 2 components, etc., Class MC vessels, etc., and ducts of radiation management equipment.

In addition to the above, there are core supports and Class 4 piping. Here, “component” is a generic name for vessels, piping, pumps, and valves; and components and supports are collectively defined as components, etc.

Regarding facilities for coping with severe accidents, etc., the component classification is mentioned in Sect. 9.7.

(2) Classification of Welded Joints

In the ASME Boiler and Pressure Vessel Code and also in several other structural codes, the welded joints are classified into A, B, C, and D according to their importance in consideration of possibilities of fracture as pressure retaining components and other factors. Examples of classification of welded joints of a vessel are shown in Fig. 9.8. In this figure, the classification of A to D that is provided as Fig. NB-3351-1 [23] in the ASME Boiler and Pressure Vessel Code is subdivided into i)–iii). The subdivision is made by the author, aiming at illustrating clearly the locations of joints.



(Note) The symbols represent the following categories.
A: joint category A C: joint category C
B: joint category B D: joint category D

Fig. 9.8 Examples of classification of welded joints of a vessel ([23], the subdivision i) – iii) is added)

- (a) Joint category A: longitudinal joints of vessel shell, pipe, or nozzle (A-i) of Fig. 9.8), joints of spherical vessel, head or flat plate (A-ii)), and circumferential welded joints between hemispherical head and vessel shell, pipe, or nozzle (A-iii)).
- (b) Joint category B: circumferential joints of vessel shell, pipe or nozzle (B-i)), and circumferential joints between non-hemispherical head other than semi-spherical heads and vessel shell, pipe, or nozzle (B-ii)).
- (c) Joint category C: joints between flange, flat plate or tube sheet, and vessel shell, pipe, or nozzle.
- (d) Joint category D: joints between nozzle, and vessel shell, pipe, nozzle, head or flat plate.

Tables 9.4, 9.5, and 9.6 show the correspondence between the importance of components and the classification of welded joints and the combinations of nondestructive testing methods for facilities for design basis events of light water nuclear power plants. Table 9.4 [21] shows testing methods that should be performed in the acceptance examination for materials. For reference purposes, this table also provides those tests, fracture toughness testing in particular, that should be conducted other than nondestructive examinations. Class 1 components, etc., are required to pass extremely stringent level of testing compared with general industrial plant components. Classes 2 and 3 components, etc., are not required to pass such stringent testing, by taking into account the required safety level of those components, and because the probability of the presence of detrimental defects in them is considered to be low based on past records, as long as they are fabricated in accordance with the material specifications and their characteristics are confirmed through mechanical testing and others. However, castings of Class 2 component are required to pass radiographic testing, which excels in detecting spherical defects or similar, since they tend to contain defects such as cavities during the casting process.

Weld examinations that should be performed are provided in Tables 9.5 and 9.6 [22]. Table 9.5 shows how to select the test method according to the importance of the vessel and piping. The descriptions in the table are only for joint category A. Table 9.6 shows how the nondestructive testing requirements for Class 1 vessels vary with joint categories. Welded joints of categories A and B in Class 1 vessels and piping are required in principle to pass both radiographic testing (RT) as a volumetric testing and surface testing (magnetic particle testing (MT) or liquid penetrant testing (PT)). For general fossil-fueled power plants, other than the cases where RT is performed on 100% of the entire weld line length (called 100% RT or full RT), RT may partially be performed on 20% of the entire length (called 20% RT as partial RT). Rules on 20% RT are applicable to longitudinal welded joints and circumferential welded joints with thickness less than 25 mm in case of austenitic stainless steel, for example. They are provided in Article 56, Item 2 of the pressure vessel structural code in the Notification No. 196 of the Ministry of Health, Labor and Welfare (April 30, 2003). The application of 20% RT requires 5% reduction in joint efficiency, compared to the 100% RT. Similar provisions are stipulated in JIS B 8265: 2017 Construction of pressure vessel—General principles, although the limit of thickness to be applied

Table 9.4 Nondestructive testing as material acceptance examinations for components, etc., of facilities for design basis events [21]

Component class	Component	Material	Nondestructive testing	Other required testing (other than testing defined in the material specification)
1	Vessel	Plate	Normal beam UT	(1) Mechanical testing (2) Fracture toughness testing: 1) Drop weight testing 2) Impact testing 3) Dynamic fracture toughness testing
		Bar	Diameter $d \leq 50$ mm : MT/PT $50 < d \leq 100$: normal beam UT (radial direction) and MT/PT $100 < d$: normal beam UT (radial and axial directions) and MT/PT	
		Pipe	Other than SG heat transfer tubes : angle beam UT (axial (ECT as an alternative) and circumferential directions) and MT/PT SG heat transfer tube : angle beam UT (circumferential direction) and ECT (angle beam UT (axial direction) as an alternative)	
		Welded pipe	Normal beam UT in the plate state. For welds, RT and MT/PT after the specified heat treatment	
		Casting	RT and MT/PT	
		Forging	Normal beam UT/angle beam UT and MT/PT	
	Piping	The same as those for Class 1 vessel		(1) Fracture toughness testing: 1) Drop weight testing 2) Impact testing
	Pump	Almost the same as those for Class 1 vessel		(1) Fracture toughness testing: 1) Drop weight testing 2) Impact testing
2	Valve	Almost the same as those for Class 1 vessel		(1) Fracture toughness testing: 1) Drop weight testing 2) Impact testing
	Supports (or, support structures)	—	Bar, and $50 < d \leq 100$ mm : MT/PT $100 < d$: normal beam UT (radial direction) and MT/PT	(1) Fracture toughness testing: 1) Drop weight testing
	Vessel	—	RT for castings (if RT is difficult, normal beam UT /angle beam UT)	(1) Fracture toughness testing: 1) Drop weight testing 2) Impact testing
	Piping	—	RT for castings	(1) Fracture toughness testing: 1) Drop weight testing 2) Impact testing
	Pump	—	Outer diameter of pipe connected with casting, d : $d < 63$ mm : Nondestructive testing not required $63 \leq d \leq 115$: MT/PT $115 < d$: RT	(1) Fracture toughness testing: 1) Drop weight testing 2) Impact testing
	Valve	—	Outer diameter of pipe connected with casting, d : $d < 63$ mm : Nondestructive testing not required $63 \leq d \leq 115$: MT/PT $115 < d$: RT	(1) Fracture toughness testing: 1) Drop weight testing 2) Impact testing
	Supports	No provision		

(continued)

Table 9.4 (continued)

Component class	Component	Material	Nondestructive testing	Other required testing (other than testing defined in the material specification)
3	Vessel	—	Excluding nondestructive testing	(1) Fracture toughness testing: 1) Drop weight testing 2) Impact testing
	Piping	—	No provision	(1) Fracture toughness testing: 1) Drop weight testing 2) Impact testing
	Pump	—	No provision	(1) Fracture toughness testing: 1) Drop weight testing 2) Impact testing
	Valve	—	No provision	(1) Fracture toughness testing: 1) Drop weight testing 2) Impact testing
	Supports	No provision		
MC	Vessel	—	Nondestructive testing not required	(1) Fracture toughness testing: Drop weight testing or impact testing
	Supports	—	No provision	(1) Fracture toughness testing: 1) Impact testing
Core support structure	Supports	Plate with thickness of 19 mm or more	Normal beam UT	(1) Fracture toughness testing: 1) Drop weight testing 2) Impact testing
		Other than plate	The same as those for Class 1 vessel. However, surface testing is not required for forged materials.	

[Note]

*1: “Normal beam UT” means ultrasonic testing using the normal beam technique.

*2 “Angle beam UT” means ultrasonic testing using the angle beam technique.

*3 “MT/PT” indicates that magnetic particle testing or liquid penetrant testing is performed. Similarly, “normal beam UT/angle beam UT” indicates that ultrasonic testing using the normal beam technique or ultrasonic testing using the angle beam technique is performed.

*4 The above table describes only representative components. For example, bolts or similar are omitted.

*5 In the Material column, “—” means that material types are not restricted. Unless material types are restricted in the column on the right, the provision applies without limiting the material to plate, bar, etc.

*6 In some cases, ductile fracture testing is not required (for example, they are not required of austenitic stainless steels and high nickel alloys).

is different from those in the Notification No. 196. However, in the case of nuclear power plants, if RT is to be conducted, it must be 100% RT.

9.5 Pressure and Leak Testing

Pressure and leak testing is a testing to check the pressure resistance and leak tightness by pressurizing to the extent not resulting in fracture and can be positioned as nondestructive testing because it is performed in a nondestructive manner. In general, the nondestructive testing methods described above are regarded as nondestructive

Table 9.5 Nondestructive testing for welds of joint category A in facilities for design basis events [22]

Component class	Component	Prescribed testing	Alternative testing
1	Vessel	RT, and MT (PT if MT is inappropriate) for welds including the adjacent base material for 13 mm on each side of the weld metal.	No provision
	Piping*	RT, and MT (PT if MT is inappropriate) for welds including the adjacent base material for 13 mm on each side of the weld metal.	No provision
2	Vessel	RT	No provision
	Piping**	RT	No provision
3	Vessel	RT	No provision
	Piping**,***	RT	No provision
MC	Vessel	RT	UT

* Limited to pipes with outer diameters over 61 mm.

** Excluding those portions of pipes with outer diameters of 61 mm or less, or connected to an open vessel, extending from the relevant vessel to the nearest stop valve. For those portions, MT (PT if MT is inappropriate) is performed in the case of Class 2 piping.

*** Pipes with wall thicknesses above 19 mm.

testing, and pressure and leak testing may be distinguished from nondestructive testing. This is probably because in order to assure the pressure resistance, etc., the purpose of the above nondestructive testing methods is to confirm that no detrimental defects are present, whereas the purpose of pressure and leak testing is to directly confirm that the part has a sufficient strength against the pressure postulated in the design and that there is no leak.

The pressure applied in pressure testing is defined for facilities for design basis events as shown in Table 9.7. In this table, the test pressure applied in so-called pneumatic testing, in which a gaseous fluid is used as the pressurizing medium, is lower than that of hydrostatic testing, in which water is used as the pressurizing medium. This is because the potential risks of using a gaseous fluid are taken into account. In other words, if a leak occurred as the worst case, the pressure would not be lowered so rapidly as water since gases are highly compressible. Therefore, it is likely that cracks progress and lead to a burst. As described in Chap. 5, a burst frequently occurs during pressure testing. If a burst occurs, it may cause a severe impact on workers, etc., in the vicinity. Therefore, pressure testing should in principle be hydrostatic testing, and pneumatic testing should be performed when hydrostatic testing is inappropriate.

In the pressure testing, pressure resistance is checked through visual examination for any anomaly such as a local bulge and elongation. Leak tightness is checked using a method such as the hydrostatic pressure testing technique in which exuding water is visually examined, the bubble leak testing in which bubble growth locations are visually examined with a soap solution applied to the outer surface, and the testing

Table 9.6 Nondestructive testing for Class 1 vessel welded joints of joint weld categories A, B, C, and D in facilities for design basis events [22]

Permissible weld category	Nondestructive testing	
	Prescribed testing	Alternative testing
1. Welds of one of (1) through (4) below (1) Welds of joint category A (2) Welds of joint category B (excl. piping for heat exchanger) (3) Welds of joint category C (excl. those giving in the item "2" below) (4) Welds of joint category D formed by full penetration welding (excl. those giving in the item "3"), other than their nozzle or weld meeting one of "a" through "e" below: a. Inner diameter of nozzle is 153 mm or less. b. Angle between nozzle axis and vessel wall is 40° or larger. c. Reinforcement of vessel openings shall be done only with the vessel wall reinforcement. d. Nozzle does not receive significant piping reaction loads. e. If a backing strip is used, it shall be removed after welding is completed.	Radiographic testing, and magnetic particle testing for portions including the adjacent base material for 13 mm on each side of the weld metal (liquid penetrant testing if magnetic particle testing is inappropriate)	—
2. Welds of joint category C that are indicated in Figures 1 to 3 below (figures are not given here)	Radiographic testing, and magnetic particle testing for portions including the adjacent base material for 13 mm on each side of the weld metal (liquid penetrant testing if magnetic particle testing is inappropriate)	—
3. Welds of joint category D formed by full penetration welding (excl. their nozzle or weld meeting one of 1(4) a through e above) that are indicated in Figures 1 to 6 below (figures are not given here)		—
4. Welds of joint category D formed by full penetration welding (excl. those given in 1(4) and 3 above) and welds formed by partial penetration welding	Magnetic particle testing (liquid penetrant testing if magnetic particle testing is inappropriate) for each portion having a thickness of 1/2 of the weld depth (13 mm if 1/2 of the weld depth exceeds 13 mm). However, for the final layer, testing must be performed to include the adjacent base materials for 13 mm on each side of the weld metal.	Ultrasonic testing, and magnetic particle testing for portions including the adjacent base material for 13 mm on each side of the weld metal (liquid penetrant testing if magnetic particle testing is inappropriate), after the completion of the welding
5. Overlay welding part formed in cases where an overlay seat is built around a hole/opening and on the nozzle surface	Ultrasonic testing and magnetic particle testing (liquid penetrant testing if magnetic particle testing is inappropriate).	—
6. Welds of pressure retaining parts (excl. those falling under 1 to 5 above) and welds of canopy seal joints	Magnetic particle testing (liquid penetrant testing if magnetic particle testing is inappropriate).	Radiographic testing or ultrasonic testing
7. Welds existing between a tube and a tube sheet (excl. those related to pressure retaining parts)	Liquid penetrant testing	—
8. Welds formed by clad welding		
9. Welds designed for mounting such important items as lugs, brackets, reinforcements, braces, stiffening rings	Magnetic particle testing or liquid penetrant testing	Radiographic testing or ultrasonic testing

[Note] In the "alternative testing" column, "—" means that the alternative testing method is not specified.

Table 9.7 Pressure to be applied in pressure testing for components of facilities for design basis events [21]

Component class	Component	Test pressure	
		Hydrostatic testing	Pneumatic testing
1	Reactor pressure vessel	Until initial fuel loading : $1.25P_D$ After initial fuel loading : $1.1P_N$	Until initial fuel loading : $1.1P_D$
	Other than the reactor pressure vessel	(1) If the testing also involves the reactor pressure vessel, the requirements on the reactor pressure vessel must be followed. (2) Other than (1) : $1.25P_D$	$1.1P_D$ However, if the testing also involves the reactor pressure vessel, this value applies until the initial fuel loading.
2	—	$1.25P_D$ However, there are exceptions applicable, for example, when the testing also involves Class 1 components.	$1.1P_D$ However, there are exceptions applicable, for example, when the testing also involves Class 1 components.
3	—	$1.25P_D$ However, there are exceptions applicable, for example, when the testing also involves Class 1 components.	$1.1P_D$ However, there are exceptions applicable, for example, when the testing also involves Class 1 components.
4	—	(No provision)	$1.1P_D$
MC	Vessel	$1.15P_D$	$1.1P_D$

[Note]

 P_D : maximum allowable working pressure P_N : normal operation pressure

method using pressure change measurement in which the pressure drop is measured after a certain period of duration time [14].

In the leak testing, it is not necessary to apply pressure as high as that in the pressure testing. Leak testing methods include the above-mentioned medium-based methods and the helium leak testing method, which is highly sensitive because the helium molecule is minute.

As a summary of technical issues, one important point [12] must be mentioned here. If a detrimental defect or similar is envisaged, nondestructive examination or pressure testing is performed to check the presence or absence of such a defect, and if it is necessary, repair is performed to ensure the required quality. You must keep in mind that one of the purposes of these testing or examinations is to reduce the manufacturing and production costs (or fabrication cost). We tend to think that performing nondestructive examination or others would increase the testing/examination expense and the manufacturing and production costs. However, if you performed appropriate testing or examination in each appropriate process of the fabrication, you would be able to find defects and other flaws at an intermediate stage, and the repair cost would not be so high at that stage in many cases. On the contrary, if you proceeded with fabrication without noticing such a defect, and performed a completion examination after all parts are assembled, the fabrication would have to be restarted from the beginning, which could require a huge repair cost. Thus, testing and inspections are

essential not only for ensuring the quality prerequisite for structures under the design code or similar, but also for reducing the manufacturing and production costs.

9.6 Examination Organization (Qualification and Certification of the Examiner)

[1] General Trend

The preceding sections focused on nondestructive examinations which are defect detection methods, as a means of examination, and described its technical aspects including the principles. When performing examination, it is important to understand not only these technical aspects but also human factors, or personal factors, described in Sect. 1.3 of Chap. 1. Matters related to safety regulation, which are personal factors, were discussed in Sect. 1.5 of Chap. 1. This section describes the trends in Japan concerning certification of examiners' skills.

In the nondestructive examinations, it is extremely important to maintain the skills of the engineer performing the nondestructive examinations at or above a certain level. This is because the reliability of the examination result depends largely not only on the examination method (non-personal factors including the equipment used in the examination), but also on the reliability of the manual for performing examination and the skills of the engineer performing the examination. If examination is performed in accordance with appropriate codes and manual, a reproducible result should be obtained in theory. However, since the flaw detection conditions may vary according to parts subject to examination even if the same examination method is used, it is difficult to standardize all factors including them. To ensure the required quality, it is practical to secure and maintain the skills of the examiner at or above a certain level.

In response to this demand, since the Japanese Society for Non-Destructive Inspection established NDIS 0601 "Rules for Qualification and Certification of Non-Destructive Testing Personnel" in 1968, Japan has strived to maintain the skills of nondestructive examiners. Meanwhile, on the international side, the ISO/TC135 (nondestructive testing) of the ISO established ISO 9712:1992 "Non-destructive testing—Qualification and certification of personnel" in 1992 with the aim of unifying the levels of engineers engaged in nondestructive examinations (quoted from Reference 23; the same applies hereinafter). ISO 9712: 1992 was revised to ISO 9712: 1999, and currently to ISO 9712: 2012 Non-destructive testing—Qualification and certification of NDT personnel. In Japan, it was decided to harmonize the Japanese Industrial Standards (JIS) with international standards, and the demand for stipulating the qualification and certification of engineers for nondestructive examination (nondestructive testing engineers) on an international level is growing in the JIS.

Thereafter, the "Industrial Standardization Operating Plan" established in 2000 proposed converting relevant international standards into JIS standards in order to internationally systematize the qualification testing and certification methods for

nondestructive testing engineers, ensure the status of those engineers on an international basis, and provide the workforce of nondestructive testing on a global scale. In line with this proposal and based on the above ISO 9712:1999 standard (the latest is the 2012 edition), JIS Z 2305 Non-destructive testing—Qualification and certification of personnel was set in April 2001 [24]. Owing to the establishment of JIS Z 2305, the accreditation system and certificate system of the Japanese Society for Non-destructive Inspection (JSNDI) (NDIS 0601 and NDIS J 001, respectively) were integrated into the JIS Z 2305-based certificate system [25].

JIS Z 2305 [24] defines common requirements for qualification and certification of NDT personnel in the industry field. The certification covers skills for one or more of the following nondestructive testing methods:

- (1) Acoustic emission testing (AT).
- (2) Eddy current testing (ET).
- (3) Infrared thermographic testing (TT).
- (4) Leak testing (Hydraulic pressure testing is excluded) (LT).
- (5) Magnetic testing (MT).
- (6) Penetrant testing (PT).
- (7) Radiographic testing (RT).
- (8) Strain gauge testing (ST).
- (9) Ultrasonic testing (UT).
- (10) Visual testing (Direct unaided visual testing and visual testing that is conducted during the application of another NDT method are excluded) (VT).

In the EU, PED 97/23/EC was enforced in May 2002, and engineers engaged in nondestructive testing on welded joints of pressure vessels (although their categories are limited) distributed or used in the EU have since been required to have special qualification certificates. In line with this, the JSNDI was working to establish a PED NDT approval system that corresponds to the EU's system. In May 2005, the JSNDI and the British Institute of Non-destructive Testing (BINDT) entered into the agreement (MOU) for PED approval. Application procedures, etc., for the PED NDT approval were determined in July 2006, and the supplement testing, which is required for applying for the PED approval, started in 2007 [26]. Thus, the certificate of PED NDT approval becomes valid in the EU member nations if it is used in conjunction with the certificate of JIS Z 2305 qualification [26].

In the USA, the following programs are well known:

- The SNT-TC-1A (Recommended practice for personnel qualification and certification in nondestructive testing) as an employer-based program.
- The ACCP (ASNT Central Certification Program) as an industry-recognized third-party certification program.

Both were set by the American Society for Nondestructive Testing (ASNT). In May 2017, the JSNDI and the ASNT entered into the Mutual Recognition Agreement (MRA), and the mutual recognition started in 2018 [27]. Note that the ACCP qualification acquisition scheme for JIS Z 2305 qualification personnel started in November 2009.

[2] Nuclear Reactor Equipment

Operator's (or licensee's) examinations on welding are required as nondestructive examinations for welds in nuclear power reactors as nuclear reactor equipment. The Nuclear Regulation Authority (NRA) has defined the requirements for the operator's examination on welding as the "Guide on the Ordinance on installation, operation, etc. of commercial nuclear power reactors" (established on June 19, 2013, and revised on February 5, 2015). This Guide is a performance-based code, and its prescriptive standard (or specification standard) is the "Rules on welding for nuclear power plants" in "Codes for nuclear power generation facilities" in "Codes for power generation facilities" of the Japan Society of Mechanical Engineers (JSME). Requirements for nondestructive testing personnel are stipulated in Item v) "nondestructive testing (nondestructive examination/inspection)" in Appendix Table 2 "Applicable matters of examination methods for operator's examination on welding, and codes/rules on welding, etc." in the above-mentioned Guide, as follows:

- (a) Qualified personnel according to JIS Z 2305 (2001) or qualified personnel according to industrial standards that are recognized as equivalent to JIS Z 2305.
- (b) The nondestructive testing personnel is required to have passed an examination in accordance with such accreditation examinations with objectivity that the accreditation schemes, accreditation procedure, etc., are verified by third parties.

The testing should be carried out according to the testing methods that are specified in the above-mentioned JSME Rules on welding for nuclear power plants and others. Items to check for defects are listed in the Guide.

9.7 Regulations on Examination/Inspection After the Fukushima Daiichi Nuclear Accident

9.7.1 Regulations on Examination/Inspection for Strengthening Safety Regulations in Response to the Fukushima Daiichi Nuclear Accident

Reflecting lessons learned from the Fukushima Daiichi nuclear accident, the safety regulations were drastically changed and strengthened. Their key points are summarized as follows. The detail is described in Chaps. 1, 5, and 7.

- (1) Tsunami countermeasures such as seawall are explicitly stipulated as the regulatory requirements, and these countermeasures are required to be designed as facilities for design basis events.

- (2) The fourth- and fifth-level safety measures as the defense-in-depth safety design concept are subject to safety regulation, although in the past, the fourth-level safety measures were practically entrusted as voluntary (or self-controlled) efforts.

Concepts of the safety measures to meet the items (1) and (2) above have been developed in line with the current global trends in safety regulations. It should be noted here that facilities for design basis events are defined as facilities that function as the first- to third-level safety measures in the defense-in-depth concept. These facilities have safety functions of preventing the occurrence of anticipated operational occurrences (AOOs) or design basis accidents, or preventing the escalation of AOOs to accidents or the accidents to severe accidents. On the other hand, facilities for coping with severe accidents, etc. are required to function as the fourth-level safety measures.

Components, etc. of facilities for coping with severe accidents, etc. are classified into severe accidents, etc. class 1 components, etc. (may be abbreviated as class SA-1 components, etc.), severe accidents, etc. class 2 components, etc. (similarly, class SA-2 components, etc.), and severe accidents, etc. class 3 components (similarly, class SA-3 components), as shown in Tables 5.13 and 5.14 in Sect. 5.10.2 of Chap. 5. “Components, etc.” is a generic term for components including vessels, piping, pumps and valves, and their support structures (or supports). Class SA-1 components, etc. are those of the Specialized Safety Facilities to cope with aircraft attack, terrorism, etc. Class SA-2 components, etc. are permanent components, etc. that are installed in the facilities for coping with severe accidents, etc., excluding the Specialized Safety Facilities. Roughly speaking, class 3 components are portable components for coping with severe accidents, etc., excluding those of the Specialized Safety Facilities.

Examination/inspection methods and their procedures to be applied to the components, etc., concerned shall be selected to achieve the quality level being required. The components, etc., are classified in accordance with their quality level to be achieved. Thus, the examination/inspection methods and their procedures vary depending on the classified level of components, etc. The quality levels of components, etc., of facilities for design basis events and these for coping with severe accidents, etc., are classified in the safety regulation as follows.

- (i) The quality levels of components, etc. of facilities for design basis events are classified according to the same classification concept as that being adopted before the Fukushima Daiichi nuclear accident, because these facilities function as the first- to third-level safety measures, which were regulated at that time.
- (ii) Table 5.14 in Chap. 5 shows the design concept for assuring the structural integrity of components, etc. of facilities for coping with severe accidents, etc. The quality levels of classes SA-1 and -2 components are set to be equivalent to that of Class 2 components of facilities for design basis events, as can be seen from this table as well as the requirements for welds in Interpretation 13 in Article 55 “Materials and structures of equipment for coping with severe accidents, etc.” in “Interpretation of the Ordinance on Technical

Standards for Commercial Nuclear Power Reactors and their Auxiliary Facilities.” The quality level of class SA-3 components can be interpreted to meet the requirement on quality level if those comply with regulatory standards for general industry products as the final product, according to the Interpretation of the aforementioned Technical Standards. For example, the Japan Electrical Manufacturers Association (JEMA) provides rules for materials and structures, pressure tests, and others of classes SA-1 and -2 valves of facilities as the prescriptive standard, referring to those for class 2 valves of facilities for design basis events. If materials and structures, pressure tests, and others of class SA-3 valves comply with the codes and standards for general industry products as the final product, those can be interpreted to meet the above-mentioned requirements for class SA-3 valves of the Technical Standards [28].

- (iii) Article 21 of the aforementioned Ordinance on Technical Standards provides rules on pressure tests that for facilities for design basis events, the hydrostatic test pressure should basically exceed the maximum allowable working pressure (or design pressure) of the component and be such a pressure that gross deformations in the component must be kept within the elastic regime for class 1 components, class 2 components, class 3 components and the containment vessel (or class MC vessel). Meanwhile, for facilities for coping with severe accidents, etc., Article 58 of the Ordinance provides rules that when the hydrostatic test is carried out at the pressure to which the component is subjected in service, classes SA-1, -2, and -3 components are required to withstand the pressure and not to cause any significant leakage.

Therefore, nondestructive examination methods and their procedures for welds in vessels and piping of classes SA-1 and -2 components should be equivalent to those for vessels and piping of class 2 components of facilities for design basis events.

9.7.2 *New Examination/Inspection Scheme After 2020*

[1] Basic Concept

Examinations at the whole stages from construction through operation and decommissioning should be performed by the operator (or licensee), while the NRA as the nuclear regulatory body confirms through the nuclear regulatory inspections that the nuclear facilities comply with the relevant technical standards. The pre-service examinations/inspections, which were conducted at the stage of construction but not at the operation stage, facility periodic examinations/inspections, and others, should be carried out by the operator. In the past before 2020, however, these examinations/inspections were done by the regulatory body even for nuclear power reactors. The past inspection scheme was slightly complicated because of differences in scheme between nuclear power reactors and testing reactors. As schematically shown in Figure 1.4 in Chap. 1, Sect. 1.5.2, Item [6], the examinations for welding are performed by the operator (or licensee) of nuclear power reactors, over which the

Ministry of Economy, Trade and Industry (METI) had supervision, and as the final inspection, the safety management review on welding was carried out by the regulatory body (actually, the Japan Nuclear Energy Safety Organization). The review was a kind of inspection (or audit) for the system of implementing the operator's examination for welding. On the other hand, for nuclear testing reactors, processing facilities, and so forth, over which the Ministry of Education, Culture, Sports, Science, and Technology (MEXT) has supervision, the regulatory body conducted the welding inspection (not the audit type inspection) as the final inspection. For example, for the High-Temperature Engineering Test Reactor (HTTR), of which one of the authors was involved in the design, safety review, and others, the welding examinations were performed by the fabricators, and then the regulatory body (at that time, the Science and Technology Agency, the predecessor of the MEXT) directly conducted the welding inspection, but the Japan Atomic Energy Research Institute (JAERI), the operator or licensee of the HTTR, did not any welding examinations/inspections at that time. Therefore, there was a concern that the JAERI as the operator could not have any concern in quality assurance of the welding. As described above, before the IRRS review, a large difference existed in examination/inspection scheme between nuclear power reactors and other than the power reactors. It may be better to proceed step by step the aforementioned reforms of examination/inspection scheme for other than nuclear power reactors.

[2] Nondestructive Examination/Inspection at the Construction Stage

Reforms in accordance with the IRRS in regard to requirements for nondestructive examinations/inspections on welding of nuclear power reactors may be summarized as follows:

- (1) Pre-service inspections that at that time were carried out by the regulatory body should be done as a part of the operator's examinations. Further, since the operator's examinations on welding are the operator's examinations to be conducted before the start of service, the operator's examinations on welding should be included in the pre-service examinations.
 - (2) It should be displayed with symbols, etc. that the operator's pre-service examinations on welding have been completed.
 - (3) The detail of the pre-service examinations on welding is basically the same as that of the operator's examinations on welding in the past and will be provided in the "Ordinance on Installation, Operation, etc. of Commercial Nuclear Power Reactors" and the "Operation Guide on Operational Safety Measures."
 - (4) The conformity of the operator's pre-service examinations, including those on welding, to the technical standards is confirmed through the nuclear regulatory inspections by the Nuclear Regulation Authority (NRA).
 - (5) The nuclear reactor facilities can only be used after confirming the conformity.
 - (6) Performance requirements on welds are prescribed in the "Ordinance on Technical Standards for Commercial Power Reactors and their Auxiliary Facilities."
- Fig. 9.9 shows the detail of the Ordinance and in what articles of the Act on

the Regulation of Nuclear Source Materials, Nuclear Fuel Materials and Reactors” the Ordinance is referred to. With regard to nondestructive examination/inspection for welds, any defect that is produced by welding is required to be detected through nondestructive testing (or examination), and then to be removed. Prescriptive (or specification) standards in accordance with the above-mentioned performance requirement include standards of academic societies and associations such as the “Rules on Welding for Nuclear Power Plants” in “Codes for Nuclear Power Generation Facilities” of the JSME. Although the JSME has not yet established any rules on welding for facilities for coping with severe accidents, etc., as mentioned in Sect. 9.7.1, rules on vessels and piping of classes SA-1 and -2 may be set, referring to those on class 2 vessels and piping of facilities for design basis events.

Concerning defects that are detected after the start of service but not at the construction stage, any crack or other defect to cause fracture is not allowed to exist in components, etc., to be designated in facilities for design basis events, as prescribed in Article 18 in the aforementioned Ordinance on Technical Standards, and correspondingly, for facilities for coping with severe accidents, etc. in the Article 56 in the Ordinance. The interpretation of the term “crack or other defect to cause fracture” is addressed by the “Interpretation of crack or other defect to cause fracture in commercial nuclear power reactors and their auxiliary facilities” (the NRA established it on August 6, 2014). The defect sizing (or to measure the size of defect) is required in order to assess the possibility of causing fracture, as mentioned above. To perform the sizing, it is required to correctly measure the dimensions of defect through ultrasonic examination methods and others and to measure the defect depth with a given accuracy. In order to meet this requirement, it is necessary to use appropriate equipment and procedures for detecting and measuring the defect, and further qualified examination/inspection personnel. Hence, performance demonstration (PD) test systems have been established not only overseas but also in Japan.

[3] Criteria for Judging Detriment of Flaw/Defect in Welds (Weld Defect Acceptance Criteria)

As shown in Fig. 9.9, the NRA provides the following Ordinances and Guides as performance requirements for welds as the acceptance criterion, on the basis of assessment result on whether the flaw is classified as a detrimental flaw, or defect, and whether the assessed part is judged to be acceptable as no defect.

The “Ordinance on Installation, Operation, etc. of Commercial Nuclear Power Reactors.”

The “Ordinance on Technical Standards for Commercial Nuclear Reactors and their Auxiliary Facilities.”

- The “Guide on the Ordinance on Installation, Operation, etc. of Commercial Nuclear Power Reactors pertaining to Operator’s Examinations for Welding of Nuclear Power Reactor Facilities.”

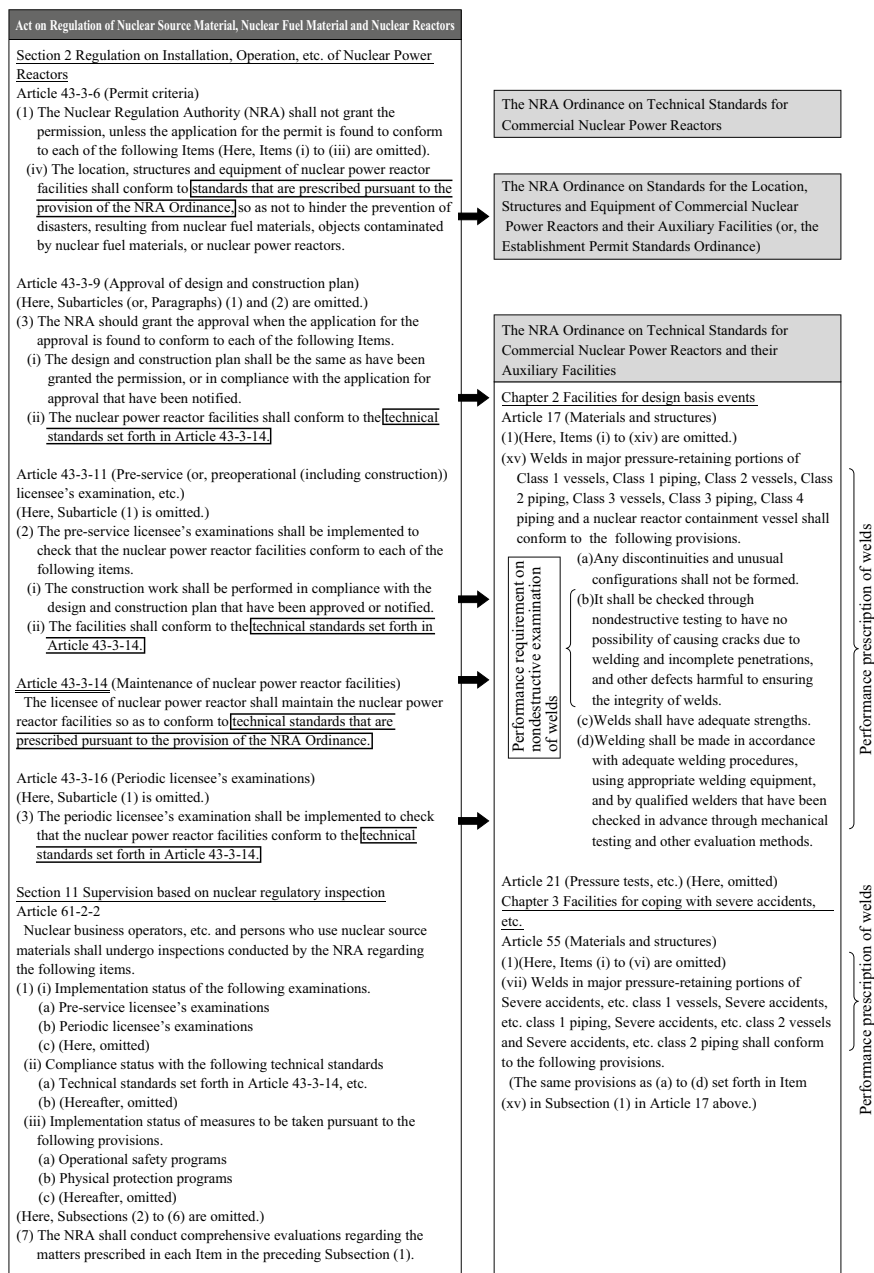


Fig. 9.9 Performance requirements on welds and their prescriptions in the Reactors Regulation Act

The following are required in the Guide:

- (i) To conduct the radiographic testing in accordance with the JSME “Rules on Welding for Nuclear Power Plants” (hereinafter, called the “Rules on Welding”) and others.
- (ii) To mainly check whether the assessment part involves a blow hole, incomplete penetration, crack, and others.
- (iii) To conform to the “Rules on Welding” and others.

In the “Rules on Welding” as the prescriptive standard, prescribed (or regular) examinations/inspections are stipulated for the following classes of vessels and piping of facilities for design basis events:

- Classes 1 vessels, MC vessel, 2 vessels, and 3 vessels (and vessels equivalent to class 3 vessels).
- Classes 1 piping, 2 piping, 3 piping (and piping equivalent to class 3 piping), and 4 piping.

The prescribed examinations/inspections are as follows:

- Radiographic examinations/inspections as volumetric nondestructive examinations/inspections for welds at the construction stage.

The acceptance criteria for radiographic examinations/inspections are as follows [22]:

- (a) A category 1 flaw shall be acceptable, as specified in Annex 4 “Classification methodology of flaw images using the radiographs” in JIS Z 3104: 1995.
- (b) If there are types 1 and 4 flaws, the following conditions shall be met. The definitions of terms category 1 and types 1, 2, 3, and 4 are given later.
 - (i) In a case when the distance between each flaw and the other flaw of types 1 and 4 next to that flaw is shorter than 25 mm, the maximum diameter of each flaw shall be no greater than $0.2t$; t is the thickness of base material (if exceeding 3.2 mm, no greater than 3.2 mm).
 - (ii) In a case when the distance between each flaw and the other flaw of type 1 adjacent to that flaw is longer than 25 mm, the maximum diameter of each flaw shall be no greater than $0.3t$ (if exceeding 6.4 mm, no greater than 6.4 mm).
 - (iii) In the above-mentioned case, the flaw or imperfection that is not counted as the flaw mark (or flaw score) in (a) above shall not be considered as a flaw.
- (c) In the range of length 12 times the thickness of base material, the distance between adjacent flaws of type 2 shall be shorter than $6L_2$; L_2 is the length of the flaw of type 2 with the longer distance, and the sum of lengths of the flaws of type 2, if these are successive and in a straight line, shall not exceed the base material thickness.

The flaw images are categorized by ranking according to the flaw mark, depending upon the thickness of base material. The flaw mark (score) is given by type, length, and number of flaws existing in the test field of vision. The category 1 is one of the categories of flaw images and can be said to be the slightest flaw. On the other hand, the flaws are classified into the following four types.

- Type 1 flaws include “round blow holes and flaws similar to them.” Such a rounded defect has an influence on structural strength due to the reduction in cross-sectional area that is caused by the flaw/defect [29].
- Type 2 flaws include “elongated slag inclusions, pipes, incomplete penetrations, and flaws similar to them.” A stress concentration caused by the type 2 flaw has an influence on structural strength [29].
- Type 3 flaws include “crack” and flaws similar to them. This type of flaw is thought to significantly reduce the structural strength due to a very large stress concentration [29].
- Type 4 flaws include “tungsten inclusions.”

In categorizing the flaw image, type 3 flaws are classified as type 4 flaws.

The detail of the aforementioned standards is not mentioned here, but an example of the acceptance criteria for flaw is given below. If the thickness of base material is over 25 mm and 50 mm or less, the test field of vision becomes 10 mm × 20 mm. The allowable limit of flaw mark for category 1 is 4, according to Table 5 in Annex 4, referred to in (a) above, under the condition of the aforementioned thickness. The flaw mark limit for the type 4 flaw is 3 if the flaw diameter is over 3.0 mm and 4.0 mm or less. Under the flaw condition mentioned above, only one defect is acceptable. However, if the thickness exceeds 4.0 mm, and then the flaw mark becomes 5. It becomes unacceptable. For the above-mentioned thickness, the flaw with a dimension of 1.0 mm or less need not be counted as flaw (mentioned in Table 4 in Annex 4).

The flaw classification for welded joints of stainless steels, heat-resistant steels, nickel alloys, etc., is provided in Annex 4 (regulations) “Classification by the image on radiograph” in JIS Z 3106:2001 Methods of radiographic examination for welded joints in stainless steel. Similar criteria for aluminum alloy and others, and titanium are given in JIS Z 3105: 2003, and JIS Z 3107: 1993, respectively.

The above-mentioned rules for acceptance criteria on nondestructive examinations/inspections for welds are those that were regulated before the new examination/inspection system was established in 2020, but remain basically unchanged in the new system, because the operator’s examinations for welding for application to nuclear power reactors before 2020 have been taken over as the operator’s pre-service examinations in the new system.

In the new examination/inspection system that has been started in 2020, the operators (or licensees) are responsible for examinations, whereas the regulatory body mainly audits the operator's examination as inspection. Clearly recognizing that the nuclear operators have the primary responsibility for ensuring the nuclear safety of their nuclear reactor facilities, it is desirable for both the operators and the regulatory body in cooperation, while interacting appropriately, to further improve the nuclear safety, and to promote more efficient use of limited resources, under the policy of performance-based, less prescriptive, and risk-informed regulations in relation to nuclear safety. There exists safety of citizens as well as local residents ahead. Further, it is expected to pave the way for revival of nuclear energy utilization.

Exercise

1. Explain the principles, advantages, and disadvantages of (a) magnetic particle examination, (b) eddy current examination, (c) liquid penetrant examination, (d) radiographic examination, and (e) ultrasonic examination, which are adopted as nondestructive examination methods for reactor materials and structures, within 200 letters for each.
2. The following statements contain errors. Correct the error(s) in each item.
 - (1) The eddy current examination method is capable of applying an eddy current deep within the test object by increasing the frequency of an alternating-current magnetic field, and therefore is suitable as a volumetric examination method for defects.
 - (2) In the ultrasonic examination method, a defect is detected using a higher frequency sound wave than that of audible sound. Normally, an ultrasonic wave with frequency 1 to 10 MHz is used. Sound waves in a solid body include a longitudinal wave, a transverse wave, and a surface wave, and a transverse wave is used most commonly in examination. However, in case of ordinary angle beam testing, a longitudinal wave is commonly used.
3. Explain the principles, detectable defects, and ways of application of the following five nondestructive test methods for reactor component materials, within 150 letters and by using at least five terms selected from the terms listed below for each test method.
 - (1) Ultrasonic testing
 - (2) Eddy current testing
 - (3) Magnetic particle testing
 - (4) Radiographic testing
 - (5) Liquid penetrant testing.

Terms: volumetric testing, surface testing, base material, magnetic material, weld, weld reinforcement, casting, angle beam technique, normal beam technique, magnetic leakage flux (magnetic leakage field), eddy current, magnetic flux, acoustic anisotropy, magnetization, probe, frequency, surface skin depth,

reference test, impedance, linear defect, indication, developer, film, magnifying glass, radiograph, magnetic particle pattern, penetrant, test liquid.

4. Explain in which part of nuclear facilities the following six nondestructive test methods are used, and for what purposes they are used, with examples.
 - (1) Ultrasonic testing
 - (2) Eddy current testing
 - (3) Magnetic particle testing
 - (4) Radiographic testing
 - (5) Liquid penetrant testing
 - (6) Leak testing.
5. Eddy current testing method is generally classified as surface testing method. When applying it to steam generator tubing, it may be regarded as one of volumetric testing methods. Explain the reason.
6. Select four items from the following nondestructive examination techniques, and then briefly explain their principle of detecting defects and detectable defects (if answer 5 or more items, disable all answers).
 - (1) Ultrasonic testing
 - (2) Eddy current testing
 - (3) Acoustic emission testing
 - (4) Radiographic testing
 - (5) Liquid penetrant testing
 - (6) Magnetic particle testing.
7. Explain deterioration events, examination techniques, repair techniques, and preventive maintenance techniques in relation to structural materials used in light water reactors, in about 100 letters for each, by using all terms listed in the parenthesis below.
 - (1) Acoustic emission testing method (measurement principle, advantage).
[In this chapter, question(s) relating to Chapter 9 are cited].
8. (Answer the following questions similar to the question 7 above)
 - (1) Acoustic emission testing method (measurement principle and advantage)
 - (2) Eddy current testing method (measurement principle, skin effect, and advantage)
 - (3) Magnetic particle testing method (measurement principle, measurement object, and advantage)
9. Select 4 items from the following nondestructive examination techniques, and then briefly explain their principles, features, and detectable flaws as effectiveness of these techniques, in particular, against flaws (what kinds of flaws occur in what parts). Regarding the parts, you can list specific parts of light water reactors and can explain general features (if answer 5 or more items, disable all answers).

- (1) Eddy current testing
 - (2) Liquid penetrant testing
 - (3) Magnetic particle testing
 - (4) Leakage magnetic flux testing
 - (5) Ultrasonic testing
 - (6) Radiographic testing.
10. Explain the following terms.
- (1) Volumetric examination and surface examination
11. As mentioned in Chap. 5, the defect-free design concept is the basic design philosophy being applied at the construction stage (or the design and construction stage). Detrimental defects that are detected by welding examinations/inspections (the operator's pre-service examinations for welding) are required to be removed, as summarized below. The radiographic examination/inspection is required to be carried out as prescribed (or regular) volumetric examination/inspection. On the other hand, the maintenance codes as mentioned in Chap. 11 provide rules for defects that are found at the stage of operation. The ultrasonic inspection method needs to be applied as the nondestructive volumetric inspection for detection and measurement of defects at this stage, because radionuclides such as fission products (FPs) are deposited on the inner surface of walls of the primary cooling system components during the operation, and due to accessibility and so on. The ultrasonic inspection method, however, has different strengths and weaknesses in detecting flaws from those of the radiographic inspection method. Explain your views on what issues should be resolved regarding the flaws to be detected, if the radiographic examination/inspection method is used as examination/inspection of flaws at the stage of construction, while the ultrasonic method is employed as inspection of flaws at the operation stage, and also on how to resolve these issues.

It should be noted here that as described in Chap. 1, the ASME Boiler and Pressure Vessel Code was revised so as to permit the use of ultrasonic examination/inspection method as the volumetric examination/inspection of flaws except for casting products at the construction stage. Since for the casting products, special attention must be paid to round flaws/defects such as blow holes, the radiographic examination/inspection method is required to apply to these products, taking into consideration that the radiographic examination method excels in detection of such round flaws.

Differences in basic philosophy regarding how to cope with flaws/defects between design/construction codes and maintenance codes

Basic philosophy

Design and construction codes; rules are developed by applying the defect-free design concept. At the design stage, allowable limits are set so as not to develop any detrimental defect under specifications for design and evaluation such as applied loading

and a number of occurrences of postulated events, in consideration of the mechanical state of component being required for each operating/service condition. At the construction stage, material processing, manufacturing, installation, etc., should be made to meet the prerequisite condition of no defect in the component material, and examinations/inspections should be carried out to confirm the prerequisite condition.

Maintenance codes; on the assumption that flaws and defects, especially cracks, may be developed in service due to “unexpected” environmental phenomena, etc., rules are provided on how to find a flaw/defect after the start of service, to measure it, and to evaluate it (that is, fitness for service) (“unexpected” is at the design stage).

	Background and requirements in design and construction codes	Background and requirements in maintenance (or, fitness for service) codes
Basic philosophy and requirements on defects	a. At the construction stage, harmful flaws (or, defects) should be removed, and then, the components can be used as basically defect-free ones. b. At the design stage, applied loads, frequencies of postulated events, etc. are limited so as not to generate any harmful defects in service, and it is not allowed to violate these design limitation. c. Assume an appropriate size of defect in consideration of examination accuracy, especially the detection lower limit, and it is required to have sufficient fracture toughness not to cause unstable fracture/non ductile fracture for the assumed defect size under service loading conditions.	In U.S.A., for example, cracks due to high-cycle fatigue and stress corrosion cracking (SCC) were found in operating nuclear power plants in 1960s and 1970s. In order to cope with this issue, it was concluded that periodic examinations/inspections should be conducted in service for detecting and sizing existing flaws. The methods, extents, etc. of examinations/inspections are determined considering the importance of components. When detecting flaws, measure the sizes and evaluate the possibility of causing unstable fracture. Repairs and replacements of flawed portions or components are required if the flaws have the possibility of causing the fracture during use.
Examinations /inspections required	i. Material acceptance examinations ii. At the stage of fabrication and installation, preservice (or, preoperational) examinations*¹ including weld examinations	i. In-service inspections (ISIs) ii. Pre-service inspections (PSIs)*¹ for obtaining information on initial condition of components for ISI.

[Note] As mentioned in the main text, do not confuse the preservice licensee’s examinations at the construction stage with the PSI for ISI.

It should be noted here that as described in Chap. 1, the ASME Boiler and Pressure Vessel Code was revised so as to permit the use of ultrasonic examination/inspection method as the volumetric examination/inspection of flaws except for casting products at the construction stage. Since for the casting products, special attention must be paid to round flaws/defects such as blow holes, the radiographic examination/inspection method is required to apply to these products, taking into consideration that the radiographic examination method excels in detection of such round flaws.

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Chapter 10

Fracture Mechanics



On the assumption that a component (material) contains a “crack,” a strength evaluation scheme is used to evaluate the fracture resistance (or integrity) of the component by predicting its fracture behavior on the basis of the mechanical properties at the tip of the crack using a continuum mechanics approach is called fracture mechanics.

In the previous chapters, the focus has been basically on equipment and components that do not contain crack. However, crack may occur and grows while equipment and components are actually in use. In such cases, fracture mechanics is applicable. The reasons that fracture mechanics is gaining attention include the increasing size of structures, the development of weld structures, the use of high-tensile materials, and severe service conditions.

This chapter describes the basic concept of fracture mechanics. Assessment of the structural integrity of components, etc., performed through the application of fracture mechanics will be described in Chap. 11.

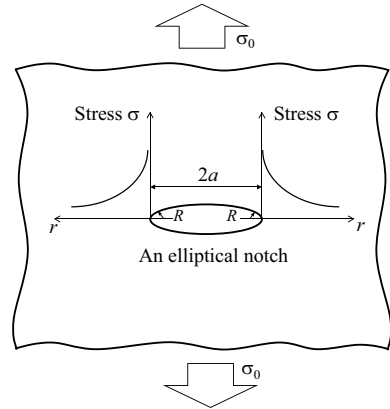
10.1 Stress Concentration

In general, when a component containing a notch is subjected an external force, high stress occurs near the tip of the notch. This is known as the stress concentration. The deeper the notch and the smaller the curvature radius of the notch tip is, the greater the magnitude of stress concentration becomes.

Figure 10.1 shows the stress distribution around a notch. In this figure, constant stress σ_0 in the vertical direction is applied at a location sufficiently far from an elliptical hole that penetrates a large plate. The lengths of the major axis and the minor axis of the elliptical hole are denoted as $2a$ and $2b$, respectively. The figure shows the stress distribution in the loading direction.

The maximum stress $\sigma_{\max}$ in this case is the stress in the vertical direction at the notch tip and is given by the following equation:

Fig. 10.1 State of stress around a notch tip



$$\sigma_{\max} = \sigma_0 \left(1 + 2\sqrt{a/R} \right) = A \cdot \sigma_0. \quad (10.1)$$

Here, the stress concentration factor A ($A = 1 + 2\sqrt{a/R}$) represents the maximum factor of stress increase caused by the presence of the notch.

The sharper the notch shape is, the greater the maximum stress $\sigma_{\max}$ becomes. Thus, a notch with a sharp notch tip, i.e., an infinitely small radius of curvature R , is a “crack” treated in fracture mechanics.

10.2 Singular Stress Field and Stress Intensity Factor at a Crack Tip

Figure 10.2 shows three independent deformation modes around a crack tip when load is applied to a cracked component.

- Mode I: crack opening type—since this is the most important mode in practice, the following description assumes mode I.
- Mode II: in-plane shear type.

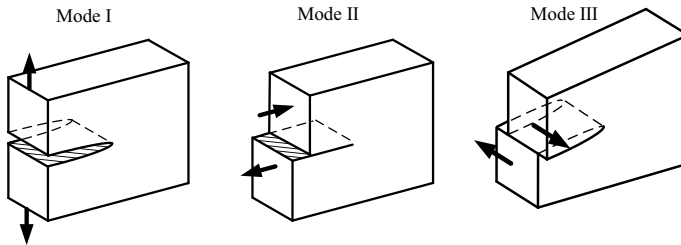
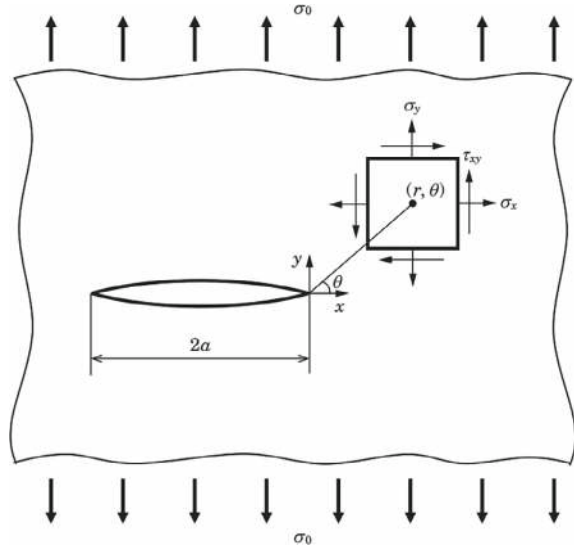


Fig. 10.2 Classification of deformation modes

Fig. 10.3 Stress field around a crack tip



- Mode III: out-of-plane shear type.

The stress distribution around a crack for the case of a constant stress σ_0 is applied in the vertical direction at a location with a sufficient distance from the crack which can be expressed as follows with reference to the notation in Fig. 10.3:

$$\sigma_{ij}(r, \theta) = \sum_{n=-1}^{\infty} A_n (\sqrt{r/a})^n f_{ij,n}(\theta) \quad (10.2)$$

In this equation, only the first term of $n = -1$ has a singularity of $r^{-1/2}$ when $r \rightarrow 0$, while the second and later terms do not have a singularity. Therefore, the stress distribution around a crack tip can be approximately expressed in the first term of Eq. (10.2) in a range, where r is sufficiently smaller than the representative dimension a of the crack.

For Mode I, the state of the stress around the crack tip is expressed by

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} = \frac{K_I}{\sqrt{2\pi r}} \cos \frac{\theta}{2} \begin{Bmatrix} 1 - \sin \frac{\theta}{2} \sin \frac{3\theta}{2} \\ 1 + \sin \frac{\theta}{2} \sin \frac{3\theta}{2} \\ \sin \frac{\theta}{2} \cos \frac{3\theta}{2} \end{Bmatrix} \quad (10.3)$$

In the above equation, K_I represents the strength of the stress field and is called as a stress intensity factor. On the x -axis ($\theta = 0, y = 0$), the following equation is obtained concerning the vertical stress σ_y :

$$K_I = \lim_{r \rightarrow 0} \sqrt{2\pi r} \cdot \sigma_y \quad (10.4)$$

A typical expression for the stress intensity factor is the following equation using the stress σ_0 at an infinite distance (i.e., a location far from the crack) and crack length a :

$$K_I = \sigma_0 \sqrt{\pi a} \cdot f(a/W) \quad (10.5)$$

Here, $f(a/W)$ (where a and W represent crack length and representative specimen size, respectively) represents a function defined by the stress and geometry. Equations of $f(a/W)$, which are analytically and empirically obtained, are provided in relevant codes, handbooks, etc.

For example, among the fracture toughness test specimens described in Sect. 10.6, the stress intensity factor for a compact tension test specimen (see Fig. 10.11a) is obtained by

$$K_I = \frac{F}{BW^{1/2}} f(a/W) \quad (10.6)$$

$$f(a/W) = \frac{(2 + a/W)(0.886 + 4.64a/W - 13.32a^2/W^2 + 14.72a^3/W^3 - 5.6a^4/W^4)}{(1 - a/W)^{3/2}}$$

Here, F represents the axial force, B represents the specimen thickness, and W represents the specimen width.

Similarly, as practical methods to calculate the stress intensity factor in cases such as a through wall crack in a flat plate, as shown in Fig. 10.4, or an axial semielliptical surface crack in a cylinder, representative calculation equations are given in codes such as the Rules on Fitness-for-Service for Nuclear Power Plants of the Japan Society of Mechanical Engineers [1] (see Sect. 11.2 of Chap. 11).

The opening displacement of a crack surface, v , is expressed by

$$v = \frac{1+\nu}{2\pi E} K_I \sqrt{2\pi r} \sin \frac{\theta}{2} \left(\frac{4}{1+\nu} - 2 \cos^2 \frac{\theta}{2} \right) \quad (10.7)$$

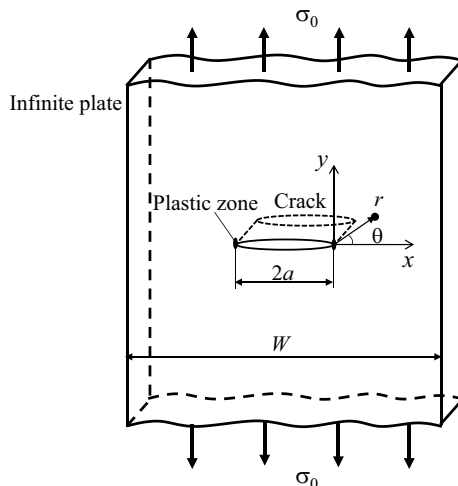
where ν is the Poisson ratio. Therefore, the opening displacement behind the crack tip on the x -axis ($-x$), where $\theta = \pi$, is given by

$$v = \frac{2}{\pi E} K_I \sqrt{2\pi(-x)} \quad (10.8)$$

The dimension of the stress intensity factor K_I is as follows, as is obvious from Eqs. (10.4)–(10.6).

$$[\text{stress}] \times [(\text{length})^{-1/2}] = [\text{force}] \times [(\text{length})^{-3/2}]$$

Fig. 10.4 Infinite plate containing a crack



It is noted that this differs from the stress concentration factor $A = 1 + 2\sqrt{a/R}$ described in Sect. 10.1, which is dimensionless.

10.3 Principle of Superposition

When a linear elastic body is subjected to n kinds of arbitrary external forces, the stress, strain, and displacement at an arbitrary point generated when each individual external forces is applied under the same conditions are proportional to each of the external forces if the displacement is very small. If these external forces are simultaneously applied, the respective stress, strain, and displacement are generated as the sum of all individual external forces. The stress intensity factor is defined by the stress or the displacement near the crack tip and is proportional to the stress or the displacement. Therefore, the principle of superposition is also applicable to the stress intensity factor.

By using the principle of superposition, it is possible to draw a useful solution through the superposition of known solutions. It is also possible to find a solution by reversely extracting only the stress required for the desired solution from the superimposed stress fields. Figure 10.5 schematically shows the principle of superposition.

As shown in Fig. 10.5a, the state of stress when a crack is present in an object can be regarded as the superposition of Fig. 10.5b, c. Here, (b) shows that stress $-\sigma$ is acting on the closed crack surface, and (c) shows that stress σ in the opposite direction is acting only on the crack surface. This suggests that you should only find the stress intensity factor in the state (c) since the stress intensity factor in the state (b), where the crack is closed, is zero.

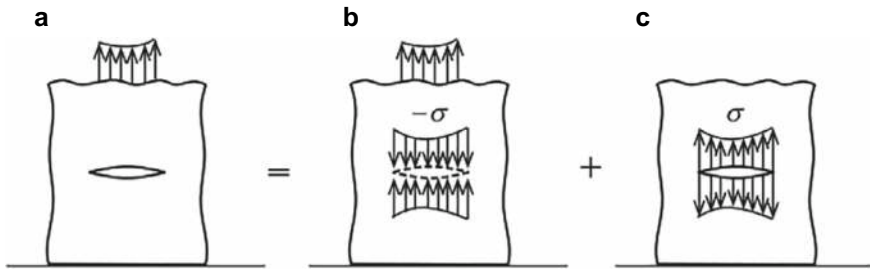


Fig. 10.5 Evaluation method based on the principle of superposition. **a** Superimposed state, **b** state of crack closed, **c** state of stress applied only

In addition, if the deformation mode of the crack surface is the same, it is possible to obtain the stress intensity factor K by calculating stress intensity factors $K_p, K_q, K_r, \dots$, for different load systems $p, q, r, \dots$, and simply superimposing them. However, if the modes differ, the principle of superimposition will not be applicable.

As an example, where superposition is actually required, Fig. 10.6 shows a case of a weld residual stress. Since the stress distribution in the region containing the crack is not uniform, it is necessary to calculate the stress intensity factor based on the principle of superposition taking into account the stress distribution. Specifically, you can obtain the stress intensity factor by first approximating the stress distribution to a high-order polynomial in a region assumed to contain a crack, determining stress intensity factors based on the stress distribution of each order, and then calculating the sum of them. In the case of Fig. 10.6, residual stress is high on the A side of the weld line, decreases as it moves away from the weld line, and tends to increase as it approaches the B side. In such a stress field, assuming that a crack exists on the A side, the stress intensity factor increases as the crack length a increases, but peaks at a certain length and around the point where the stress becomes negative (compressive), the stress intensity factor decreases due to the effect of compressive stress if the crack is deeper.

Figure 10.7 shows the stress caused by internal pressure, thermal load, etc., generated in a thick-walled component. The linear stress component can be divided into membrane stress and bending stress. The portion off the linear stress component is defined as the peak stress. It is possible to obtain the stress intensity factor for the total stress by applying the equation of the stress intensity factor for each stress to calculate each stress intensity factor and adding up these factors (see Sect. 11.2.3 of Chap. 11).

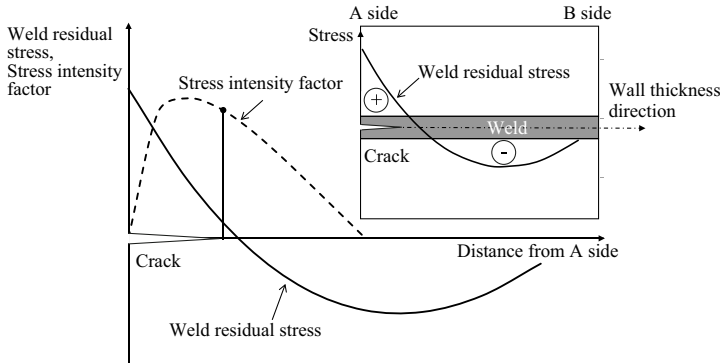


Fig. 10.6 Stress intensity factor and weld residual stress

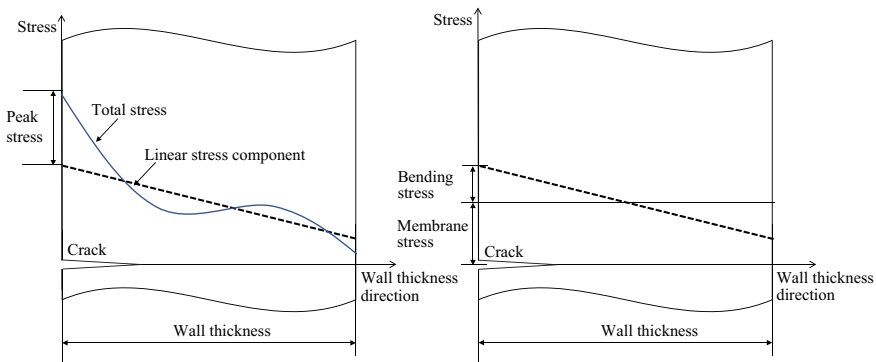


Fig. 10.7 Stress distribution in the wall thickness direction of a thick-walled component

10.4 Similarity Rule: Notches and Cracks

Figure 10.8 shows a standard object containing a crack, and a similar object with a size n times larger. This section discusses the relationship between these objects. The apostrophe (') indicates the symbol for the n times larger object.

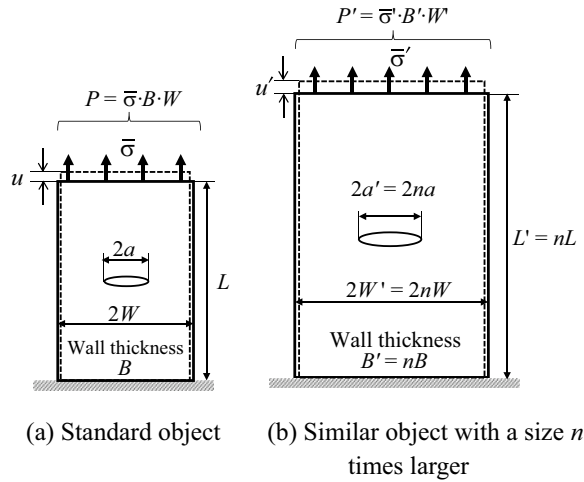
In order to assure the stresses of the corresponding points of the two objects to be equal, the following equations are applied:

$$P^* = \bar{\sigma}' \cdot W' = \bar{\sigma} \cdot nW = nP^*, \quad \bar{u}' = n\bar{u} \quad (10.9)$$

In these equations, P^* represents the load per plate thickness. Therefore, the portions not related to the presence of the crack can be regarded as being proportional to the dimensional ratio n .

However, since the stress distribution of the crack tip is proportional to $1/\sqrt{r}$, the relationship between the stress intensity factors of the standard object and the similar

Fig. 10.8 Comparison between similar objects both containing a crack.
a Standard object, **b** similar object with a size n times larger



object can be expressed as $K' = \sqrt{n}K$. Therefore, in order to assure that both stress intensity factors are equal at the corresponding crack tips, the following relationship must hold true.

$$P^{*'} = \sqrt{n} P^*, \quad \bar{u}' = \sqrt{n} \bar{u} \quad (10.10)$$

In this way, it should be noted that the similarity rule for similar objects is different for the case with and without cracks.

10.5 Plastic Region of Crack Tip

The preceding sections assumed that linear elastic bodies that show only elastic behaviors are the subject of study. In the following, a small-scale yielding condition that occurs near the crack tip, plastic region correction, and plastic regions in the cases of plane stress and plane strain are explained as more realistic behaviors.

Because stress is very high at the crack tip, in reality, the material yields at the crack tip as well as a limited region in front of the crack tip, and plastic deformation occurs. Figure 10.9 shows the mechanism of plastic deformation.

If two symmetrical pairs of slip systems move at a crack tip and slip alternately, the crack tip will open due to plastic deformation as shown in Fig. 10.9b, c, and the originally sharp crack tip will change from A to A' and then to A''. This change is called blunting. In reality, blunting occurs continuously, and the crack tip becomes rounded as a result [2].

If the size of the zone where plastic deformation occurs is sufficiently smaller than the extent of the singular stress field of the crack tip, the singular stress field

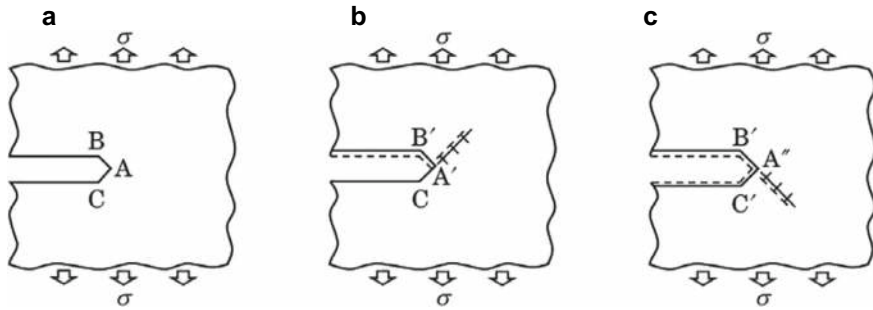


Fig. 10.9 Mechanism of slip plane separation and blunting at crack tip

surrounding this zone (plastic zone) can be expressed by the stress intensity factor. Such state is called “small-scale yielding condition,” for which the above-described concept of linear elastic fracture mechanics is effective.

Under small-scale yielding condition, the plastic zone dimension r_p at the crack tip is sufficiently smaller than the crack length a . Therefore, the following equations can be expressed for the plane stress state and the plane strain state, respectively.

$$r_p = \frac{1}{2\pi} \left(\frac{K_I}{\sigma_{ys}} \right)^2 \dots \text{plane stress}, \quad r_p = \frac{1}{2\pi} \left(\frac{K_I}{\lambda \sigma_{ys}} \right)^2 \quad \text{plane strain} \quad (10.11)$$

Here, K_I represents the Mode I stress intensity factor, σ_{ys} represents yield stress of material; λ represents a plastic constraint factor: $\lambda = \sigma_1/\sigma_y$, σ_1 represents maximum principal stress, respectively.

As the plastic zone is produced at the crack tip, the singular stress field moves. Therefore, it is necessary to correct the stress intensity factor. Let us consider the correction of a plastic zone in a plane stress state. In the plastic zone, the stress is equal to the yield stress, and outside the plastic zone, the stress distribution is equal to the stress distribution for a hypothetical crack of length a^* . If the plastic zone dimension at the crack tip is ω , the plastic zone correction is given by

$$a^* = a + r_p, \quad \omega \approx 2r_p = \frac{1}{\pi} \left(\frac{K_I^*}{\sigma_{ys}} \right)^2 \quad (10.12)$$

Here, $r_p \ll a$, $K^* \approx K$ and $\omega \ll a$.

In the case of plane strain, the following Irwin's equation for the plastic constraint factor λ is substituted into Eq. (10.11) [3].

$$\lambda = \sqrt{2\sqrt{2}} \quad (10.13)$$

Therefore, the following equation can be obtained for the plastic zone dimensions in the plane strain state.

$$r_p = \frac{1}{4\sqrt{2}\pi} \left(\frac{K_I}{\sigma_{ys}} \right)^2 \tag{10.14}$$

In the case of a crack in a specimen of a certain thickness, the relationship between the stress state in the thickness direction and the size of the plastic zone is considered. The stress state along the crack front edge of the specimen is plane stress near the surface and approaches plane strain in the interior. Therefore, the plastic zone is represented by a shape that is larger near the surface (Fig. 10.10a) and smaller inside (Fig. 10.10b) [4].

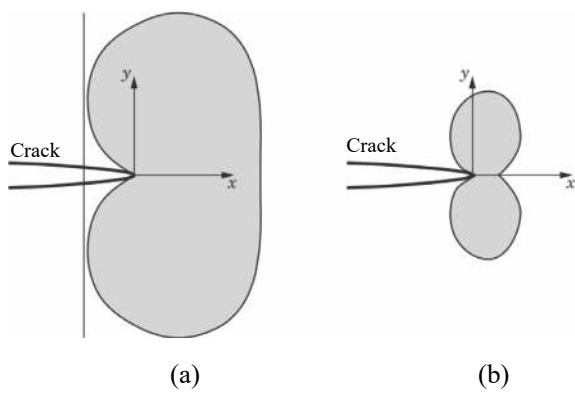


Fig. 10.10 Plastic zone shape along the crack front of a through wall crack in a plate specimen. **a** Near the surface: plane stress, **b** inside the plate: plane strain

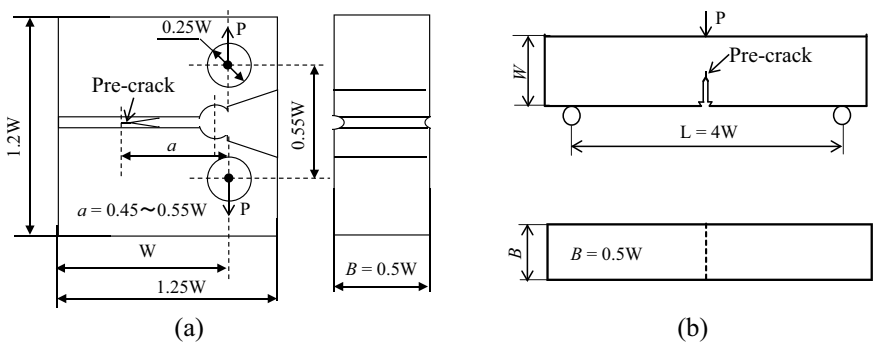


Fig. 10.11 Typical fracture toughness test specimens [5]. **a** Compact tension test specimen **b** Three-point bending test specimen

10.6 Condition of Brittle Fracture Initiation and Fracture Toughness Value

10.6.1 Fracture Initiation Condition and Fracture Toughness

For Mode I, the condition for the initiation of crack growth (fracture for brittle materials) is expressed by the following equation using stress intensity factor:

$$K_I \geq K_{Ic} \quad (10.15)$$

where K_{Ic} is defined as the value corresponding to the initiation of fracture, using a fracture toughness test specimen, depending upon the temperature and loading type of the material to be tested. The initiation of crack growth, or the critical value of fracture initiation, defined in this manner is called fracture toughness. Fracture toughness is usually obtained by loading a specimen of a specified shape with a crack (pre-crack) of a certain length in advance by cyclic loading (fatigue loading) and then subjecting the specimen to a specified load to fracture it.

Figure 10.11 shows typical test specimens [5]. The length of the pre-crack is required to be about 50% of the specimen width.

- (1) Compact tension (CT) specimen: This is the most standard specimen. Two pins run through its upper and lower holes, and tensile plus bending loads are applied to the middle plane where the crack exists.
- (2) Three-point bend test specimen: This is a typical test specimen used for bars. With a crack produced at the center of the specimen and both ends of the specimen supported by rollers, a compressive load is applied to the central part of the specimen above the crack.

The fracture toughness K_{Ic} is determined as the stress intensity factor K_I , calculated from the load at the time of fracture, by actually causing brittle fracture using the above specimens under conditions such as material, loading mode, and temperature that would cause brittle fracture.

10.6.2 Evaluation of Fracture Toughness

[1] Validity of Fracture Toughness Value

At the crack tip, even if a plastic zone is formed, a stress intensity factor is valid when the small-scale yielding condition is satisfied. Therefore, a criterion corresponding to the small-scale yielding condition is required. Generally, the following equation

is used as the dimensional condition of a specimen for obtaining fracture toughness in a plane strain state (plane strain fracture toughness) K_{Ic} [5]:

$$B, a \geq 2.5 \left(\frac{K_{Ic}}{\sigma_y} \right)^2 \quad (10.16)$$

Here, B represents the thickness of the specimen and a represents the crack length. Since for general fracture toughness specimen $a \approx 0.5W$, the ligament length of the crack section is expressed as $b = W - a \approx a$. This means that, in comparison to Eq. (10.11), the thickness of the specimen and the ligament length must be more than ten times greater than the size of the plastic zone at the crack tip.

[2] Representative Examples of K_{Ic}

The American Society of Mechanical Engineers (ASME) Boiler & Pressure Vessel Code, Section XI, sets a K_{Ic} curve so that it passes through the lower bound of all K_{Ic} data acquired using a large number of materials for reactor pressure vessels [6]. The K_{Ic} curve is organized with respect to the reference temperature RT_{NDT} for each steel. The RT_{NDT} is the reference temperature for ferritic steels and is an index of the so-called ductile-to-brittle transition temperature, which is determined from the drop weight test and Charpy impact test (see Sects. 11.2, 11.3 and 11.4 of Chap. 11).

For ferritic steel, brittle fracture occurs in a low-temperature range and ductile fracture in a high-temperature range. Therefore, the fracture toughness tends to increase with temperature. In general, such a behavior is called the “ductile–brittle transition,” and the fracture toughness in the temperature range where this transition occurs (ductile–brittle transition temperature range) is regarded as an important material property in view of preventing brittle fracture. In consideration of this property, the above K_{Ic} curve is generally expressed as an exponential function of temperature.

[3] Effect of Specimen Thickness on Fracture Toughness

Many researchers have built databases regarding the variation of fracture toughness data for various specimen thicknesses. An example of a representative fracture toughness database in Europe is provided in Reference 7. This database contains fracture toughness values in the ductile–brittle transition temperature range with the specimen thickness (B) measuring from 12.5 mm to 100 mm for the same material. However, because some small specimens do not satisfy the above Eq. (10.16), K_{Jc} , calculated from the J -integral (J_c) at the point of brittle fracture after exhibiting elastic–plastic behavior, is used as the fracture toughness value instead of K_{Ic} (see Sect. 10.8 for J_c , $K_{Jc} = \sqrt{EJ_c/(1 - \nu^2)}$). The higher the temperature, the more widely the fracture toughness tends to vary as affected by the specimen thickness. In addition, it has been shown that the thinner the specimen, the less likely brittle fracture is to occur due to the lowering of the constraint (i.e., approaching the plane stress state) at the same temperature, and therefore, the more likely ductile fracture is to occur.

Therefore, if the specimen to be evaluated for the fracture toughness is thinner than the steel material that is actually used in components, it is important that the

specimen should satisfy the K_{Ic} criterion described above. If it doesn't satisfy the criterion, it is necessary to take a measure such as correcting the specimen thickness.

[4] Dynamic Fracture Toughness K_{Id}

The fracture toughness testing described above is performed statically (i.e., at a very low speed). However, if a fracture toughness value is measured at a very fast displacement speed while taking into account the state of the load applied to real components, the value is called a dynamic fracture toughness value. In general, if the displacement speed is high in low-alloy steel, the fracture toughness value is comparatively small. This means that the yield stress increases and the ductility decreases when the strain rate increases; just like that the yield stress increases and the ductility and fracture toughness decrease when the test temperature lowers. Therefore, the ductile–brittle transition temperature at high-strain rate shifts to a higher-temperature range.

[5] Crack Arrest Fracture Toughness K_{Ia}

Crack arrest fracture toughness K_{Ia} is obtained by experimentally measuring the arrest behavior of a crack that has started to propagate brittlely. A notch is machined in the weld bead as a brittle material, and a brittle crack is generated from the notch. A temperature gradient or a displacement-controlled load is applied to the specimen to create a condition in which brittle crack propagation is easily stopped. K_{Ia} is determined experimentally by calculating the stress intensity factor at the point where crack propagation stops.

[6] Ductile–Brittle Transition and the Lower Bound Curve of Fracture Toughness: K_{IR} Curve (Lower Bound Envelope Curve of K_{Ic} , K_{Id} , and K_{Ia})

ASME Boiler & Pressure Vessel Code, Section III, sets a reference fracture toughness K_{IR} curve by summarizing and plotting the above-mentioned fracture toughness values and drawing a curve that passes the lower bound of all the fracture toughness data [7]. The purpose of using the K_{IR} curve is to conservatively define fracture toughness for various types of loads. The method of normalizing various steel materials with reference to the reference temperature RT_{NDT} , as described for the K_{Ic} curve, was originally adopted for the K_{IR} curve. The K_{IR} curve was applied to structural design as a curve applicable to Class 1 components.

[7] Recent Trend: Fracture Toughness Master Curve Method

The fracture toughness master curve method [8] was developed at VTT of Finland. The method is currently adopted by the American Society for Testing and Materials (ASTM) and the Structural Integrity Assessment Procedure (SINTAP), a European standard. It is used for evaluating the fracture toughness of ferritic steel in the ductile–brittle transition temperature range.

This method is used to estimate the temperature dependence, specimen size effect, and variability of fracture toughness (K_{Ic}) in the ductile-to-brittle transition temperature range from fracture toughness specimens of 10 to 100 mm in thickness. This method is called the fracture toughness master curve method because it is possible

to express the temperature dependence of fracture toughness with one average curve (master curve) by compiling the data of a large number of ferritic steel materials. This method has already been standardized in the USA and Japan [9, 10].

When the specimen thickness is corrected to 25 mm, the master curve corresponding to the median value of fracture toughness is expressed by the following equation:

$$K_{Jc(\text{med})} = 30 + 70 \exp \{0.019(T - T_o)\} \quad (10.17)$$

In this equation, T_o is a reference temperature for the master curve of material tested. In the correction of a fracture toughness value according to the difference in the specimen thickness, the fracture toughness value is treated as proportional to the 1/4th power of the specimen thickness based on the weakest link theory. In addition, the variation of fracture toughness values follows the Weibull distribution, which is also based on the weakest link theory.

10.7 Ductile Fracture

As described in Sect. 3.2.1 of Chap. 3, a fracture that is accompanied by significant elongation, reduction in area, and other deformation before the final fracture (rupture) is called ductile fracture.

If fracture does not occur under a small-scale yielding condition and the plastic zone rapidly grows, the singular stress field will disappear and the stress intensity factor K will lose its meaning as a fracture mechanics parameter. For such a large-scale yielding condition, J -integral or crack opening displacement (COD) are usually effective.

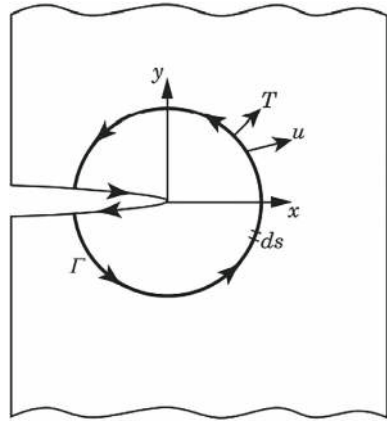
J -integral, proposed by Rice [11], is a line integral that does not depend on the path, and is expressed as

$$J = \int_{\Gamma} \left(W dy - T \frac{\partial u}{\partial x} ds \right) \quad (10.18)$$

where Γ is an arbitrary anticlockwise integral path that encircles a crack such as that shown in Fig. 10.12, s is the length along the integral path, and W is the strain energy density defined by the following equation. T and u represent the stress and displacement vectors on Γ , respectively.

$$W = \int_0^{\varepsilon_{ij}} \sigma_{ij} d\varepsilon_{ij} \quad (10.19)$$

When there is no volume force or inertial force, this J -integral becomes a potential energy release rate if the specimen is an elastic-plastic body compliant with the total strain theory.

Fig. 10.12 *J*-integral path

Let us assume that the stress–strain relationship is expressed by the exponential hardening law $\varepsilon = A\sigma^n$. Then, a singular stress field called the HRR (acronym of Hutchinson, Rice, and Rosengren who developed this theory) field [12, 13], expressed by the following equation, is generated:

$$\sigma_{ij} = \left(\frac{J}{AI_n r} \right)^{\frac{1}{n+1}} \tilde{\sigma}_{ij}(\theta) \quad (10.20)$$

Here, I_n is a factor dependent only on n , and $\tilde{\sigma}_{ij}(\theta)$ is an eigenfunction that provides distribution concerning θ .

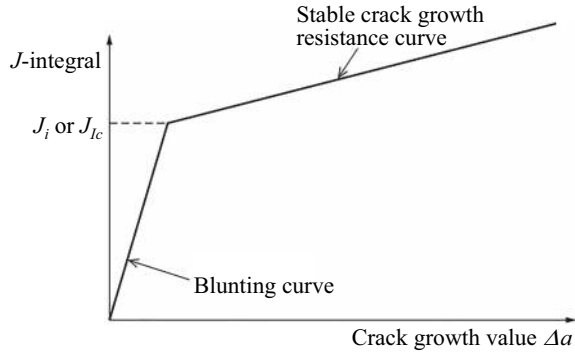
Thus, under certain restrictions, *J*-integral can be representative of the mechanical state near a crack tip, such as stress and strain, as K in a linear elastic case.

10.8 Occurrence and Stable Growth of Ductile Crack

Concerning the evaluation of ductile fracture behavior using the *J*-integral method described above, this section describes J_{Ic} , which is a ductile fracture parameter for the initiation of a ductile crack, and the *J*-*R* curve (or the *J*- Δa curve), which characterizes the stable growth of a ductile crack. The section also mentions the net-section collapse failure criterion applied to austenitic stainless steel pipes, etc., as an engineering method to evaluate ductile fracture.

As explained in Sect. 10.8, *J*-integral is effective as a fracture mechanics parameter for ductile fracture if large-scale yielding occurs at a crack tip. Figure 10.13 shows a concept of a crack growth resistance curve (*J*-*R* curve) based on *J*-integral. In the initial stage of deformation, the crack tip is blunted and the *J*-integral value increases along the blunting line. A ductile crack initiates at the blunted crack tip,

Fig. 10.13 Concept of J - R curve



and the crack starts to grow. The J -integral value at this point is generally expressed as J_i or J_{Ic} . After stable crack propagation, the J -integral increases slowly and follows the resistance curve.

For materials with high ductility, high toughness, and low yield stress such as austenitic stainless steel, the ultimate fracture will occur in the form of plastic collapse in accordance with stress criteria even if a crack is present. As a simple and practical evaluation method to determine the applied load when a crack starts to grow or penetrates through a plate in such a case, the net-section collapse failure criterion given by the following equation was proposed by Kanninen et al. [14] and is currently used widely.

$$\sigma_{\text{net}} = \sigma_f \quad (10.21)$$

Here, σ_{net} is the net-section stress (i.e., stress on the cross section excluding the crack), and σ_f is the flow stress of the material (usually $\sigma_f = (\sigma_y + \sigma_u)/2$).

In addition, an evaluation method that is able to consider both ductile and brittle fracture at the same time has been developed in the UK, and it is generally called the R6 method [15]. This method was then improved and is extensively adopted as a two-parameter method in the ASME code and JSME Rules on Fitness-for-Service.

10.9 Time-Dependent Fracture and Crack Growth

Fracture of a brittle material or plastic instability of a ductile material, that we have dealt with so far, occurs almost instantaneously as a result of load increase. Therefore, such a phenomenon is called time-independent fracture. By contrast, fracture may also occur after a certain period of time when a load is repeatedly applied or maintained for a certain period of time as shown in Fig. 10.14. This is called time-dependent fracture [4], which includes fatigue fracture, stress corrosion cracking and creep fracture.

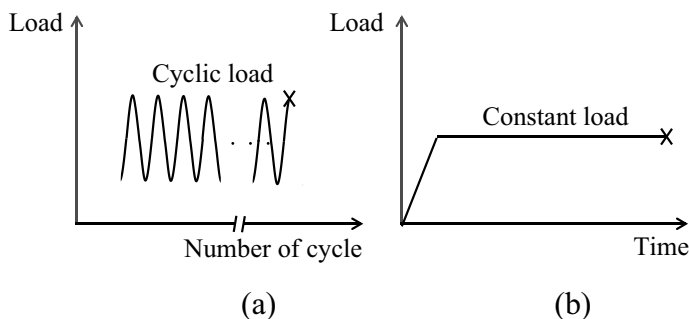


Fig. 10.14 Types of loads that result in time-dependent fracture. **a** Cyclic load, **b** constant load

Fatigue fracture occurs when a load is repeated regularly or a load fluctuates irregularly, as shown in Fig. 10.14a. If the load has reached the yield stress, the whole material deforms. As a result, a new surface is exposed due to slipping, the surface area increases, and elongation occurs. If the stress is at or below the yield stress, the whole material will not deform plastically. However, local slipping occurs preferentially in a phase or crystal grain with the weakest structure on the material surface, oriented in a direction at which slipping would easily occur. If this stress is repeated, a geometric change occurs on the surface as a result of slipping. The slipping under repeated stress results in a corrugated surface. Once a corrugated surface is formed, stress concentration occurs in the concave region, and its depth increases inward owing to irreversible slipping that occurs preferentially in this region. This is how fatigue cracking occurs. If there is a macroscopic notch, flaw, or inclusion on the material surface, stress concentration occurs because the surface is corrugated from the beginning and thus fatigue cracking occurs easily.

Corrosion is a surface dissolving phenomenon where a battery is locally formed in an active environment with a metallic material surface being an anode. If a constant load is maintained as shown in Fig. 10.14b, dissolution at the anode may locally occur because of the cooperative action of tensile stress and develop into a crack, and the crack may further grow owing to anodic dissolution of the crack tip. This is called stress corrosion cracking (SCC). SCC occurs under a particular combination of stress, environment, and material. For example, it occurs when dissolved oxygen concentration is high in an aqueous environment of a light water reactor, and when weld residual stress is high near a weld of austenitic stainless steel.

If the melting point of a metallic material is expressed in absolute temperature, thermal activation is significant and a large number of atomic vacancies are formed inside the material in the temperature range at or above approximately 50% of the melting point. If a constant load is maintained, deformation will occur due to diffusion of atomic vacancies to the grain boundaries. This is called creep deformation. The

diffusion of atomic vacancies produces minute vacancies at the same time and serves to cause a crack and its growth, leading to creep fracture.

Through various experiments based on fracture mechanics, empirical equations concerning the crack growth rate in the above-mentioned time-dependent fracture have been proposed. The following shows commonly used empirical equations for fatigue crack growth, stress corrosion cracking, and creep crack growth.

(1) Fatigue Crack Growth (Paris' Law)

$$\frac{da}{dN} = C(\Delta K)^m \quad (10.22)$$

where a is the crack length, N is the number of cyclic load repetition, ΔK is the range of the applied stress intensity factor, and C and m are material constants.

When subjected to cyclic loading, the crack growth rate is expressed in terms of the range of the applied stress intensity factor ΔK . The amount of crack growth for one cyclic load, Δa , can be obtained from $C(\Delta K)^m$. For a large number of repetitions, the amount of crack growth can be evaluated by using Eq. (10.22).

(2) Stress Corrosion Cracking

$$\frac{da}{dt} = f(K, \text{env.}) = C'K^n \quad (10.23)$$

where t is time, and f is a function of stress intensity factor (K) and aqueous environment (env.). As described in Sect. 11.5.4 of Chap. 11, stress corrosion cracking is generally expressed as shown in the rightmost side of the equation. In this case, C' and n are constants that depend on the material, the environment, and the K value.

The crack growth rate is expressed by the value of K . Since the crack grows with time, the K value also changes with time. The crack growth behavior is evaluated by calculating the K value and the crack growth rate at a constant short interval and integrating the calculated values.

(3) Creep Crack Growth

$$\frac{da}{dt} = f(C^*, T) = A_C(C^*)^m \quad (10.24)$$

Creep occurs at high temperatures, and the crack growth caused by the creep is expressed by Eq. (10.24). Here, C^* represents the fracture mechanics parameter at which the creep crack propagates, T represents the temperature of the respective region, and A_C and m are constants that depend on the material and the temperature.

10.10 Probabilistic Fracture Mechanics

Probabilistic fracture mechanics (PFM) is an engineering approach for evaluating failure probability by taking into account the uncertainties and variations of load conditions and various parameters, as opposed to the deterministic fracture mechanics described above. The concept of PFM is shown in Fig. 10.15.

10.10.1 Method for Obtaining the Failure Probability of Components

In deterministic fracture mechanics, structural integrity assessment is generally conducted based on the use of safety factors or on conservative loads and resistances. On the other hand, in PFM, the failure probability is calculated by taking into account the uncertainty of load and resistance, and structural integrity is ensured by keeping the failure probability below a certain value. PFM methodology can be applied to areas such as reliability engineering, safety assessment, optimal design, and risk-informed assessment. Although a method to obtain the failure probability of an equipment by statistically surveying past failure records is also possible, it cannot be used unless there are sufficient long-term records of use and failure cases of the equipment. For this reason, a method of calculating the failure probability by using a numerical analysis method based on the fracture mechanism has been proposed.

The PFM methodology started to be studied in the late 1970s and began to play an important role in the safety assessments of reactor structural components in the 1980s. In recent years, especially in Europe and the USA, it has been widely used in regulatory and plant maintenance activities.

10.10.2 Analysis Models

The main analysis models are shown below.

- Modeling of components: material, geometry and crack size.
- Stress analysis: steady/unsteady load.

Fig. 10.15 Concept of probabilistic fracture mechanics

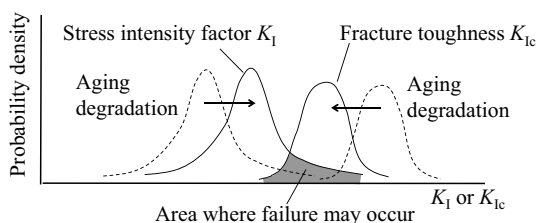
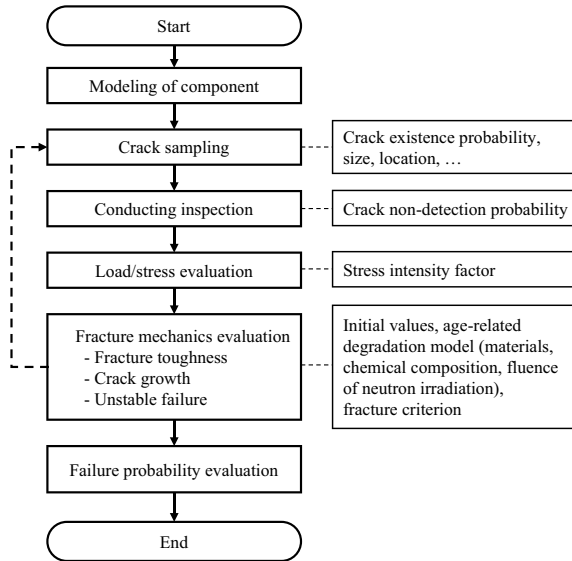


Fig. 10.16 Example of flowchart for probabilistic fracture mechanics analysis based on the Monte Carlo method



- Crack detection through inspection: non-detection probability, accuracy of crack sizing.
- Fracture mechanics analysis: stress intensity factor, crack growth analysis, fracture toughness, fracture criterion.
- Failure evaluation: comparison with allowable crack size, judgment of leak by crack penetration through the wall.

The material constants used in the crack growth analysis and fracture evaluation are treated as probabilistic variables (load may also be treated probabilistically). Together with the non-detection probability in the inspection, the failure probability is calculated with probabilistic methods such as those shown below.

- Numerical integration method: method of discretizing failure probability (integrated value of probability that the stress intensity factor exceeds the fracture toughness value) and integrating the results
- Monte Carlo method: method of obtaining failure probability from M/N , where M is the failure number in N trials. In each trial, values of probability variables are randomly changed in accordance with the probability distribution. An example flowchart is shown in Fig. 10.16.

10.10.3 Representative Example of PFM Analysis Code

A number of analysis codes have been developed as shown below with the aim of applying PFM methods to reactor structural components. For their characteristics, please refer to the respective references.

- PRAISE [16]: A PFM analysis code developed by Lawrence Livermore National Laboratory (LLNL) in the USA for light-water reactor piping based on the Monte Carlo method, including consideration of seismic loads.
- VISA-II [17]: A PFM analysis code developed by Pacific Northwest National Laboratory (PNNL) in the USA for reactor pressure vessels based on the Monte Carlo method considering pressurized thermal shock (PTS).
- OCA-P [18]: A PFM analysis code developed by Oak Ridge National Laboratory (ORNL) in the USA for reactor pressure vessels based on the Monte Carlo method for the case of PTS.
- COVASTOL [19]: A LOCA probability analysis code developed by the EC based on the numerical integration method.
- PASCAL [20]: A PFM analysis code developed by the Japan Atomic Energy Agency (JAEA) based on the Monte Carlo method for the case of non-steady loading including PTS on reactor pressure vessels.
- FAVOR [21]: A PFM analysis code developed by ORNL in the USA for reactor pressure vessels related to the revision of PTS rules in the USA.

10.10.4 Examples of Probabilistic Distribution Models of Material Constants

- Fracture toughness value: Weibull distribution [22].
- Initial crack size: exponential distribution [23].
- Crack growth rate: lognormal distribution.
- Tensile strength: Weibull distribution or normal distribution.
- Crack non-detection probability: error function or exponential distribution.

Exercises

1. Indicate the dimensional requirements of the specimen for determining the plane strain fracture toughness K_{Ic} . Also, using a compact tension specimen, indicate the dimensions of the specimen required to obtain K_{Ic} of 100 [MPa m] for a low-alloy steel with yield strength $\sigma_y = 500$ [MPa]
2. Briefly describe the procedure for obtaining plane strain fracture toughness K_{Ic} using a fracture toughness test specimen.
3. Present the words/phrases or equations that fit in (a) to (h) of the following sentences.

When a component containing a notch is subjected to an external tensile force, a large stress is produced near the notch bottom. This is called (a). The deeper the notch and the smaller the curvature radius ρ of the notch bottom, the greater the magnitude of (a) becomes. A notch with an infinitely small radius ρ of curvature at the tip of the notch is called (b) in fracture mechanics.

A fracture mechanics parameter that represents the stress field of a crack tip is called (c). This parameter is given by (Equation d). The unit of (c) is (e). In general, the condition for fracture of a component containing a crack under a monotonically increasing load is given by (Equation f).

If a crack grows due to a cyclic load, the crack growth rate is given by (Equation g) using (c). By contrast, crack growth under a constant load, as is the case with stress corrosion cracking, is given by (Equation h) as time-dependent crack growth.

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Chapter 11

Structural Integrity Assessment



In nuclear power plants, the codes and standards for design and manufacture were also applied to the plants during operation to confirm the integrity of the structures and components. In other words, the structures and components of nuclear power plants were required to be flawless as shipped from factories. For this reason, if a flaw was indicated in inspection, repairment or replacement would be conducted following the codes and standards used for reactor design and manufacture. On the other hand, in the USA, the ASME code for boiler and pressure vessel was introduced for in-service inspection in the 1970s, in which the flaw evaluation methods were stipulated for flaws detected during inspection. The code adopted the concept of fracture mechanics described in Chap. 10.

To prevent the recurrence of the fraud problems like that were revealed at nuclear power plants in August 2002 in Japan, the Electricity Utilities Industry Law was revised in December 2002, which requires the electric utilities to carry out utility periodical inspection. In addition, when a flaw is detected in a safety-related component, the licensee is obliged to assess the structural integrity of the component, record and retain the result of assessment, and report it to the government of Japan. This is so-called the integrity assessment system introduced as a maintenance standard, allowing the facility to be continuously used as long as the safety-related functions remain intact even if aging occurs in the component during its service period.

In this chapter, the concept of the integrity assessment system introduced as a maintenance standard is described.

11.1 Methods of Securing Structural Integrity

The assurance of structural integrity is performed at each stage such as design, testing, manufacturing, in-service, and reflection of trouble countermeasures. Here, “structural integrity” and “functional maintainability” are defined as follows:

- **Structural integrity:** The specified geometry must be satisfied (maintained). When this is compromised, failure is defined to occur (cracking, rupture, permanent deformation, etc.). However, the occurrence of failure depends on the definition of integrity. For example, the existence of a flaw is allowed if the definition of integrity allows its existence.
- **Functional maintainability:** The conditions imposed to fulfill the specified performance must be satisfied.

In regulatory technical standards for structures of Japan, the main articles related to structural integrity are as follows:

- (1) Safety systems, materials, and structures (in stages of design, testing, and manufacture → the requirement for fracture toughness).
- (2) Pressure-resistant test and leak test (in stages of pre-service and in-service).
- (3) Surveillance tests (in stages of pre-service and in-service).

The details are explained below:

(1) **Requirement for fracture toughness**

Fracture toughness based on fracture mechanics is required to prevent non-ductile fracture (i.e., brittle fracture). This provision will substantially ensure structural integrity.

The principle here is that the stress intensity factor should not exceed the fracture toughness (in other words, the fracture toughness should be higher than the anticipated stress intensity factor). In general, ferritic steel materials have low fracture toughness at low temperatures and high fracture toughness at high temperatures. Therefore, limiting the lowest service temperature assures the fracture toughness to be high enough during the usage at temperatures higher than the limit, which, as a result, secures the structural integrity.

(2) **Pressure-resistant tests**

These tests confirm the functions of “pressure resistance” and “air-tightness.”

A pressure-resistant test confirms whether or not the margin is sufficient for the pressure-resisting strength as it is expected in the design.

- For example, in case of conducting hydrostatic tests for Class 1 components, before the fuel is loaded, the pressure will be set as 1.25 times of the highest service pressure, and after the fuel is loaded, the pressure will be set as 1.1 times of the pressure during normal operation.

A leak test will then check the existence of small defects after the strength is confirmed.

- It is not necessary to apply a high pressure exceeding the highest service pressure. However, in the leak test using air pressure, since the confirmation is made visually, consideration is required to minimize the possibility of occurrence of injuries in case that the pressure retaining part breaks. Therefore, in a leak test using air pressure, the pressure applied is lowered to the highest service pressure. In

addition, the reactor containment vessel undergoes two leak tests, one at the time of completion and the other before fuel loading. The leak rate test is also performed during a periodical inspection using a pressure equal to 0.9 times of the highest service pressure to confirm the absence of any significant leak.

(3) Surveillance tests

It is obligatory to install surveillance specimens for ferritic components of light water reactor pressure vessels for power generation that are subject to changes in mechanical properties due to neutron irradiation during their service life and that may interfere with their operation, in order to periodically investigate and evaluate the changes due to irradiation.

Article 22 of the Regulations Concerning Technical Standards for Commercial Power Reactors and Their Auxiliary Facilities (issued by Nuclear Regulation Authority of Japan on June 8, 2018) specifies the “surveillance specimen” as follows:

“Surveillance specimen:” The interior of a vessel belonging to a facility subject to the design standard, the material of which is likely to be significantly deteriorated by irradiation with neutrons of 1 MeV or more, shall be installed with surveillance specimens as specified below so that the effects of irradiation can be confirmed in order to prevent brittle fracture under the assumed operating conditions of the vessel.

1. The material of the surveillance specimen shall have a manufacture history equivalent to that of the material of the neutron-irradiated area of the vessel.
2. The number of required surveillance specimens shall be such that the change in the mechanical strength and the fracture toughness of the vessel material can be confirmed by taking them out for testing after the start of use of the vessel.
3. The surveillance specimens shall be arranged so that they are subjected to conditions equivalent to those of the neutron spectrum, neutron fluence, and temperature history to which the materials of the vessel in the neutron irradiation zone are subjected.

The Japan Society of Mechanical Engineers’ Codes for Nuclear Power Generation Facilities—Rules on Design and Construction for Nuclear Power Plants [2] (hereinafter referred to as “Design and Construction Code”)—includes provisions on surveillance test. The code stipulates the needs of surveillance specimens for base metals and weld metals, excluding specimens for heat-affected zones.

Regarding the procedure of surveillance tests, the Method of Surveillance Test on Structural Materials of Nuclear Reactors, Nuclear Power Section, Japan Electric Association Code [5] (see Sect. 11.3 of JEAC 4201.) has been used. It describes surveillance test plans, surveillance test methods, and commentaries on them. These surveillance test methods have been developed with reference to US standards such as the ASTM International Standard E185.

The core region of a reactor pressure vessel (RPV), where the maximum neutron fluence is expected to exceed $1 \times 10^{17} \text{ n/cm}^2$ ($E > 1 \text{ MeV}$) on the inner surface of the vessel at the end of the operation period, is considered to be an area where the material may be significantly degraded by neutron irradiation.

The method to determine operational conditions on the basis of data obtained from surveillance tests should be in compliance with the Japan Electric Association Code JEAC 4206, Method of Confirmation Test of the Fracture Toughness for Nuclear Power Plant Components [6] (see Sect. 11.4).

Specifically, surveillance specimens (tensile test specimens and impact test specimens) loaded inside a reactor pressure vessel are taken out for testing several times during the service life to determine the reference temperature which adjusts to the neutron irradiation effects. The following is a summary of the aforementioned association codes that are specified for structural integrity assessment or structural integrity assurance.

11.2 Structural Integrity Assessment Methodology

11.2.1 Requirement for Fracture Toughness

This requirement sets a limit to the lowest service temperature for ferritic steel materials exhibiting low fracture toughness at low temperatures to prevent non-ductile fracture, which secures the fracture toughness to be sufficiently high at temperatures above the limit. Specific methods to satisfy the requirement are shown below. In this section, materials with a thickness of less than 16 mm and bolt materials with a nominal diameter of less than 25 mm are excluded.

The first step is to determine the reference temperature R (or RT_{NDT}). To do this, the drop weight test and the Charpy impact test are conducted as described later.

It is required to meet the relationship between the K value and the temperature (see Eq. 11.1) by applying the reference temperature.

- Stress intensity factor K for a postulated semi-elliptic surface flaw with a depth of 25% of the plate thickness and a length of 1.5 times of the plate thickness.
- Reference fracture toughness K_{IR} (corresponding to the left side of the following equation and indicating the lower bound fracture toughness of ferritic steel materials used in reactor pressure vessels).

$$29.43 + 1.344 \exp[0.0261(T - R + 88.9)] > K \quad (11.1)$$

Regarding the required value of the reference temperature (R or RT_{NDT}) for ferritic steel materials of a Class 1 vessel, the previous technical standard concerning structures, Notification 501, gives the required value in the form of an inequality between the stress intensity factor and the reference fracture toughness expressed as a function of the reference temperature, as shown in Eq. (11.1). On the other hand, design and construction code, the use of which is currently approved as part of the Government's regulatory standards, gives the required value in the rewritten form of an inequality between the reference temperature

and the temperature expressed as a function of the stress intensity factor, as shown in the next section.

11.2.2 Method of Determining the Required RT_{NDT} Value in the JSME Design and Construction Code

(1) Method of calculating the stress intensity factor K_{Iapp}

A surface flaw with a sharp semi-elliptical shape and perpendicular to the maximum stress direction is assumed as the maximum postulated flaw. The size of the postulated flaw is as described below, where t [mm] denotes the thickness of the component.

- Cross section with a thickness range of $100 \text{ mm} \leq t \leq 300 \text{ mm}$.
 - → Semi-elliptical flaw with a depth of 25% of the plate thickness and a length of 1.5 times of the plate thickness.
- Cross section with a thickness range of $t > 300 \text{ mm}$.
 - → Postulated flaw for a 300 mm thick cross section.
- Cross section with a thickness range of $60 \text{ mm} \leq t < 100 \text{ mm}$.
 - → Postulated flaw for a 100 mm thick cross section.

(2) Method of calculating K_{Iapp}

$$K_{Iapp} = S_F \cdot K_{Ip} + K_{Iq} \quad (11.2)$$

where K_{Ip} and K_{Iq} are stress intensity factors for the primary and the secondary stresses, respectively, being calculated by the method shown in the Mandatory Appendix Attachment 4-A of the code in consideration of the postulated flaw and the applied stress.

For the value of S_F , 1.5 is used for the pressure-resistant test and 2 is used for Service Levels A and B.

(3) Method of determining the required value of RT_{NDT}

$$RT_{NDT} \leq T - \frac{1}{0.036} \ln \left(\frac{K_{Iapp} - 36.48}{22.78} \right) \quad (11.3)$$

For the region with a thickness of 65 mm or less: $RT_{NDT} \leq T - 33 \text{ } ^\circ\text{C}$.

11.2.3 Examples of Calculation of the Stress Intensity Factor K_I

The general equation of the stress intensity factor is expressed as below referring to Eq. (10.4):

$$Q = 1 + 4.593(a/l)^{1.65} \quad (11.4)$$

$$K_I = M\sigma\sqrt{\pi a/Q} \quad \text{or} \quad K_I = AG\sqrt{\pi a/Q}$$

where Q denotes the flaw shape parameter for a semi-elliptical surface flaw. Related to the method of calculating the stress intensity factor, the overview of the Mandatory Appendix Attachment 4-A of the Design and Construction Code is shown below. For the reader's information, the overview is based on the 2016 edition of the code, and it is recommended to refer to the latest edition of the code.

(1) Solution in ASME B&PV Code, Section III, Appendix G [1]

Method for a flaw in a plate subject to membrane stress, bending stress, and stress arising from the radial temperature gradient.

- From the diagram, M_m for the membrane stress, M_b for the bending stress, and M_t for the stress arising from the radial temperature gradient are determined, and by multiplying each of them by its corresponding stress and summing up the results, K_I is calculated.

(2) Solution in ASME B&PV Code, Section XI, Appendix A

Method for a semi-elliptical surface flaw in a plate subject to a stress of third-order polynomial distribution.

- As shown in Eq. (11.5), the coefficients G_i ($i = 0$ to 3) at the evaluation points (the deepest point and the surface point) are obtained from coefficients shown in table, and by multiplying them by the factors A_i ($i = 0$ to 3) in the stress distribution polynomial for each order and summing them up, K_I is obtained. In Eq. (11.5), A_p is the pressure at flaw surface.

$$K_I = \{(A_0 + A_p)G_0 + A_1G_1 + A_2G_2 + A_3G_3\}\sqrt{\pi a/Q} \quad (11.5)$$

In the case of linearly approximation of applied stress to calculate K_I on the basis of membrane stress and bending stress.

- As shown in Eq. (11.6), K_I is calculated from membrane stress σ_m and bending stress σ_b , as well as the G_0 and G_1 used in Eq. (11.5).

$$K_I = \{(\sigma_m + A_p)M_m + \sigma_b M_b\} \sqrt{\pi a/Q} \quad (11.6)$$

where M_m and M_b are calculated as shown in Eq. (11.7). t is the plate thickness.

$$M_m = G_0 \quad (11.7)$$

$$M_b = G_0 - 2 \left(\frac{a}{t} \right) G_1$$

(3) Raju and Newman's Solution

Method for a semi-elliptical surface flaw in a plate subject to membrane stress and bending stress.

- For each of the deepest point A and the surface point B of a flaw, K_I is calculated from the equations for f which is corresponding to M in item (1) for membrane stress and bending stress.

(4) Shiratori's Solution

Method for a semi-elliptical surface flaw in a plate subject to a third-order polynomial stress distribution, and for an axial semi-elliptical surface flaw in a cylinder subject to internal pressure and a third-order polynomial stress distribution.

- The coefficients K_i for the polynomial stress distribution and the coefficients F_i at the K_I evaluation points are obtained from a table to calculate K_I .

(5) Zahoor, Raju and Newman, and Tan and Fenner's Solution

Method for an axial semi-elliptical surface flaw in a cylinder subject to internal pressure and a third-order polynomial stress distribution.

- The coefficients σ_i for the polynomial stress distribution and the coefficients G_i at the evaluation points are obtained from a table to calculate K_I .

(6) Example for a Flaw at a Corner on the Inner Surface of a Nozzle

The solution based on the solution of Paris & Sih and the solution for a 1/4 flaw are given. However, thermal stress is not considered in the solutions.

11.3 Surveillance Test

The overview of the method of surveillance test for structural materials of nuclear reactors is shown below. In this section, JEAC 4201-2004 is referred to. In the 2007 edition of the code, a new embrittlement prediction method and rule for long-term surveillance test plan are stipulated. The concept of the assessment of neutron irradiation embrittlement using surveillance specimens is shown in Fig. 11.1.

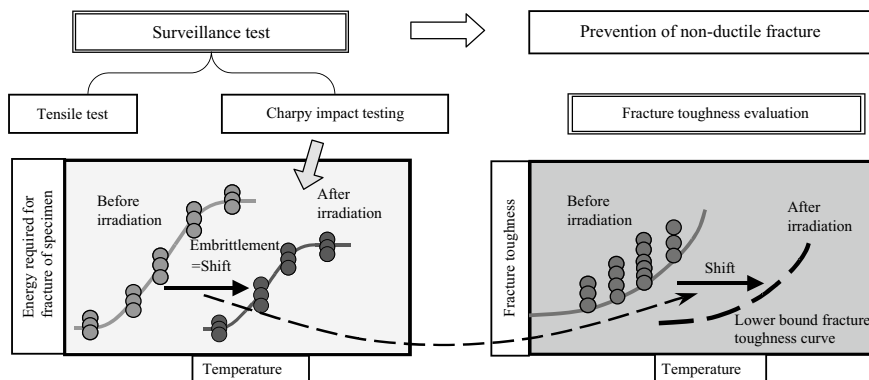


Fig. 11.1 Concept of the evaluation of neutron irradiation embrittlement using surveillance specimens

(1) Rules for surveillance specimens

- A reactor pressure vessel that is subjected to neutron fluence of 1×10^{17} n/cm² for the neutrons with energy greater than 1 MeV and there is the possibility of significant material degradation shall be installed with surveillance specimens inside.
- Surveillance specimens are placed at locations where the specimens could approximately have the same neutron flux, energy spectrum, neutron fluence, and temperature history as the structural materials.
- JEAC 4201 specifies the materials, number, type, and irradiation conditions of specimens in detail.
 - Number of tension test specimens: 3 (base metal, weld metal); number of impact test specimens: 12 (base metal, weld metal, heat-affected zone; however, 15 or more specimens are required before irradiation), manual for specimen sampling, number of test capsules, time to take out, etc.

(2) Surveillance test plan

- Material to be tested: It is selected from the base metal and the weld metal (in the circumferential or axial direction, and only the base metal in the case of a forged material) so that its adjusted RT_{NDT} value at the end of service lifetime is anticipated to be the highest.
- Test specimens: The tension and impact test specimens are based on the JIS standards. The location of sampling is at 1/4 of the plate thickness and is perpendicular to the main processing direction or the weld line.
- Irradiation condition: The test specimen should be irradiated at a position as similar as possible to that of the structural materials. The addition of accelerated irradiation is possible to be conducted to the specimens. A neutron dosimeter and a temperature monitor are necessary. The test capsule is filled with inert gas. The number of capsules and the time to take them out are

determined on the basis of the predicted amount of the reference temperature shift ΔRT_{NDT} (When the ΔRT_{NDT} value is expected to be larger, a larger number of capsules are needed).

(3) Method of surveillance test

- Measurement of mechanical properties: The tension and Charpy impact tests are in accordance with the JIS standards. In the Charpy impact test, the temperatures (the transition temperature) corresponding to a lateral expansion of 0.90 mm, absorbed energy values of 41 and 68 J, and a ratio of ductile fracture surface of 50% are determined, and the upper shelf energy is determined as well.
- Neutron irradiation fluence evaluation: On the basis of the analysis method and values measured by a neutron dosimeter, the neutron fluence irradiated with energy greater than 1 MeV is evaluated.
- Evaluation of the irradiation effect: The change in the tensile properties, the shift of the transition temperature, and the amount of decrease in the upper shelf energy due to neutron irradiation are determined. The adjusted RT_{NDT} value is obtained by adding ΔRT_{NDT} to the RT_{NDT} value obtained before irradiation.
- Prediction of the amount of reference temperature shift and the decrease rate of the upper shelf energy due to the neutron irradiation.

[Embrittlement prediction equation in Japan].

ΔRT_{NDT} can be calculated from the contents of chemical compositions (Cu, Ni, P, and Si) of the base metal and weld metal, and the fluence of neutron irradiation. Appendix B of the 2007 edition stipulates the method of predicting irradiation embrittlement, by which a calculated ΔRT_{NDT} value is determined on the basis of the Cu and Ni contents, the nominal irradiation temperature of the irradiated region, and neutron flux data.

[Prediction of the decrease rate of the upper shelf energy due to neutron irradiation].

On the basis of the technique stipulated in JEAC4201, the value of the upper shelf energy (USE) at the time of evaluation (adjusted USE value) is predicted. When the predicted value falls below 68 J, the upper shelf fracture toughness J_{mat} is predicted on the basis of JEAC 4206. In this calculation, J_{mat} is determined in correspondence to the base metal and weld metal, the Service Levels A, B, C, and D, and the evaluation temperature. For the Service Levels A, B, and C, the upper shelf fracture toughness $J_{2.5}$ when the ductile crack has grown 2.5 mm is used.

11.4 Method of Confirmation Tests of the Fracture Toughness of Nuclear Power Plant Components

This section outlines the method of confirmation tests of the fracture toughness on the basis of the Rules of the Japan Electric Association, JEAC4206-2004 and JEAC4206-2007.

(1) Purpose and method of the test

- To prevent non-ductile fracture, the validity of fracture toughness is confirmed.
- Components to be covered are safety-related facilities among Class 1 components, Class MC components, Class 2 components, and Class 3 vessels and piping.
- Materials covered: The materials covered are base metal, weld metal, and weld heat-affected zone belonging to carbon steel, low-alloy steel, and non-austenitic stainless steel among the components.

(2) Definitions of terms

- Fracture toughness, reference temperature RT_{NDT} , ΔRT_{NDT} , adjusted RT_{NDT} value, upper shelf energy.

[Method of determining the reference temperature].

First, the nil-ductile transition temperature (T_{NDT}) is determined by the drop weight test. In this case, when two of the specimens are not broken at a certain temperature, a 5 °C lower temperature is defined as the T_{NDT} . When one of them is not broken, a second test is conducted and the temperature 5 °C lower than the temperature at which four of the specimens are not broken is defined as the T_{NDT} .

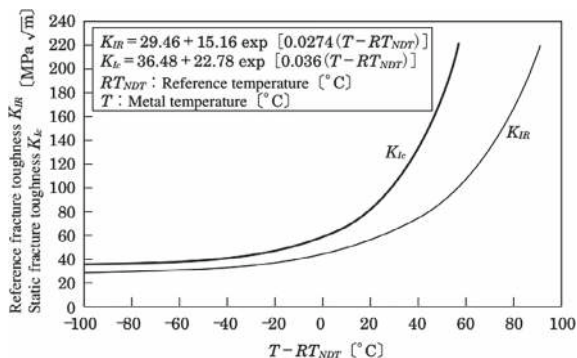
Charpy impact test is conducted at a temperature not exceeding the temperature 33 °C higher than the T_{NDT} . When three specimens satisfy the conditions of having an absorbed energy of 68 J or more and a lateral expansion of 0.90 mm or more in this test, the T_{NDT} is regarded as the reference temperature RT_{NDT} . Moreover, when the average of the three specimens satisfies the above conditions or when one of the specimens exhibits values of 54 J or more and 0.75 mm or more, the T_{NDT} is regarded as the reference temperature RT_{NDT} if a second test on two of them satisfies the above conditions.

When the conditions are not satisfied, Charpy impact test is conducted at a higher temperature. If the conditions are satisfied in that impact test, the temperature 33 °C lower than the test temperature is defined as the reference temperature RT_{NDT} .

(3) Class 1 components

- Acceptance criterion: Prevention of non-ductile fracture on the basis of fracture mechanics and other considerations.

Fig. 11.2 Relationship between the reference fracture toughness K_{IR} , the static fracture toughness K_{IC} and $T-RT_{NDT}$ for single-pass bead (Based on the equations in JEAC4206-2007)



Conditions for core region materials that are newly constructed:

- Upper shelf energy: 102 J or higher before irradiation and 68 J or higher at the end of the service lifetime
- Required value of RT_{NDT} (reference temperature $RT_{NDT} + \Delta RT_{NDT}$): less than 93 °C

$$RT_{NDT} \leq T - \frac{1}{0.0274} \ln \left(\frac{2K_{Ip} + K_{Iq} - 29.46}{15.16} \right) \quad (11.8)$$

Regarding the fracture toughness of a vessel material in the Service Levels A and B, the relationships between K_{IR} and RT_{NDT} are stipulated for the cases of a drop weight test based on the single-pass bead method and a drop weight test based on the two-pass bead method (Fig. 11.2).

When making assessment on reactor pressure vessels for the Service Levels A and B, the following static fracture toughness K_{IC} is used as the fracture toughness of the materials:

$$K_{IC} = 36.48 + 22.78 \exp[0.036(T - RT_{NDT})] \quad (11.9)$$

Here, the adjusted RT_{NDT} value is used as RT_{NDT} for the materials of core region.

Regarding the acceptance criteria, the requirements on pressure and temperature limitation under the Service Levels A and B (excluding the pressure-resistant and the leak tests) shall meet the following equation:

$$2K_{Ip} + K_{Iq} < K_{IC} \quad (11.10)$$

The requirements on pressure and temperature limitations in the pressure-resistant and leak tests shall meet the following equation:

$$1.5K_{Ip} + K_{Iq} < K_{IC} \quad (11.11)$$

- Requirements on pressure and temperature in reactor pressure vessels

Table 11.1 shows examples of requirements on pressure and temperature limitations corresponding to the Service Levels A and B and pressure of a reactor pressure vessel as well as examples of requirements on the lowest service temperature.

Furthermore, after the JEAC 4206 was published in 2007, it underwent a major revision in 2016. After the major revision, the applicable range of JEAC4206-2016 [7] became the confirmation of the validity of fracture toughness for ductile fracture and non-ductile fracture during the in-service period for reactor pressure vessels. In the revision, the JEAC 4206 is comprehensively revised that the method to determine the fracture toughness reference temperature T_0 is added, and other latest findings about fracture mechanics of structural integrity assessment are also added.

11.5 Integrity Assessment System

11.5.1 Case Where a Flaw Is Detected in Inspection

(1) Integrity assessment system

The flow and the concept of the integrity assessment system are shown in Fig. 11.3.

- It was made obligatory for utility to assess the integrity of a component when a flaw was found during a utility's periodic inspection and to report the assessment result to the regulatory authority. In other words, the method of assessing the integrity of a component when a flaw was found was established explicitly as a rule.
- Components covered: Vessels and piping (except the flanges, the joint part and the seal part, the heat transfer tube of steam generator) among components comprising reactor coolant pressure boundary (Class 1 component), and the structural supporting part of the core region (the core shroud, the shroud support ring, shroud support).
- Assessment method: As the regulatory review standard, the Japan Society of Mechanical Engineers' Codes for Nuclear Power Generation Facilities—Rules on Fitness-for-Service for Nuclear Power Plants [3] (hereinafter referred to as Fitness-for-Service Code), 2008 edition has been technologically reviewed for its technical validity. The Fitness-for-Service Code will continue to be technologically reviewed in accordance with revisions to the code to be used as a regulatory standard.

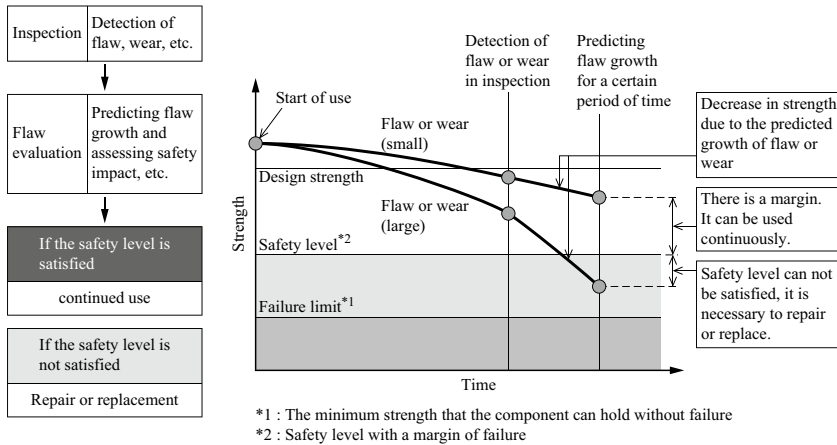
(2) Concept behind integrity assessment (see Fig. 11.3)

- The safety level is set with a certain margin in consideration of the failure limit of a structure concerned.

Table 11.1 Examples of pressure and temperature requirements of reactor pressure vessels

Operating condition		Pressure* ¹	Requirements for pressure and temperature limitations	Requirement for the lowest service temperature
Service Levels A and B (excluding the pressure-resistant and the leak tests)	Core in subcritical state	$\leq 20\%$	Pressure and temperature limitations determined by JEAC 4206 Annex A or equivalent method	Highest $RT_{NDT} + 33\text{ }^{\circ}\text{C}$ of the material in the part where the bolt tightening load acts
	Core in subcritical state	$> 20\%$	As above	The higher one of “highest $RT_{NDT} + 67\text{ }^{\circ}\text{C}$ of the material in the part where the bolt tightening load acts” and “highest RT_{NDT} of the material in the part where the bolt tightening load does not act”
	Core in critical state	$\leq 20\%$	Add $22\text{ }^{\circ}\text{C}$ to the temperature of pressure and temperature limitations obtained by JEAC 4206 Annex A or equivalent method	The higher one of “temperature requirements for the pressure-resistant and the leak tests” and “highest $RT_{NDT} + 22\text{ }^{\circ}\text{C}$ of the material in the part where the bolt tightening load acts” However, for boiling water reactor pressure vessels operated within the normal water level, the requirement “highest $RT_{NDT} + 33\text{ }^{\circ}\text{C}$ of the material in the part where the bolt tightening load acts” should be satisfied
	Core in critical state	$> 20\%$	As above	The higher one of “temperature requirements for the pressure-resistant and the leak tests” and “highest $RT_{NDT} + 89\text{ }^{\circ}\text{C}$ of the material in the part where the bolt tightening load acts”

Note * 1: Ratio to the pressure of pre-service pressure-resistant test
(Based on Table-FB-4100-1 of JEAC 4206-2007)



(a) Flow of integrity assessment

(b) Method of integrity assessment (conceptual diagram)

Fig. 11.3 The flow and the concept of the integrity assessment (Based on the website of the Nuclear Regulation Authority [8])

- When a structure is designed, it is generally designed with a margin corresponding to the safety level.
- Occurrence of flaw after service causes the structural strength of components to decrease.
- Prediction of the flaw growth value in a given period (in five years, for example).
- Continued use is allowed if the structural strength, which is determined on an engineering basis from predicted flaw size, satisfies the safety level. If the safety level is not met, repair or replacement will be regarded as necessary.
- Even if continued use is judged as allowable, the flaw growth should be checked by inspection within the predicted period during which the safety level is met (continued inspection).

11.5.2 Overview of the Flaw Evaluation Method in the Fitness-for-Service Code

The overview consists of excerpts from the document of the technical review of the Nuclear and Industrial Safety Agency for 2000 edition of the JSME Fitness-for-Service Code. From the 2002 edition of the Fitness-for-Service Code, the components covered are revised to Class 1 to 3 components, Class MC components, supporting structures, and core internals. Furthermore, the 2004 edition added the Repair/Replacement Section. In this way, the Fitness-for-Service Code formulated by the Japan Society of Mechanical Engineers will be revised at intervals of a few years

reflecting the progress in engineering levels. This section describes the overview of the Evaluation Section of Fitness-for-Service Code.

[1] Overview of the scope of the flaw evaluation

The Fitness-for-Service Code specifies the safety levels necessary to maintain the integrity of a component when a flaw occurs after the service. Using the ASME B&PV Code Section XI as its basis, the code specifies methods of flaw evaluation (evaluation of the flaw shape and flaw growth), acceptable standards (the flaw size, in consideration of safety factors, for the size limit at which unstable fracture actually occurs), and other matters.

Specifically, Class 1 components covered by the Fitness-for-Service Code are ferritic steel vessels, ferritic steel pipes, and austenitic stainless steel pipes. The flaw evaluation is to be conducted in the following two steps (Fig. 11.4).

(1) First step of flaw evaluation (flaw characterization for detected indication and allowable flaw)

Based on the indication detected during the in-service inspection (ISI) for different components, flaw characterization is conducted to determine the flaw orientation,

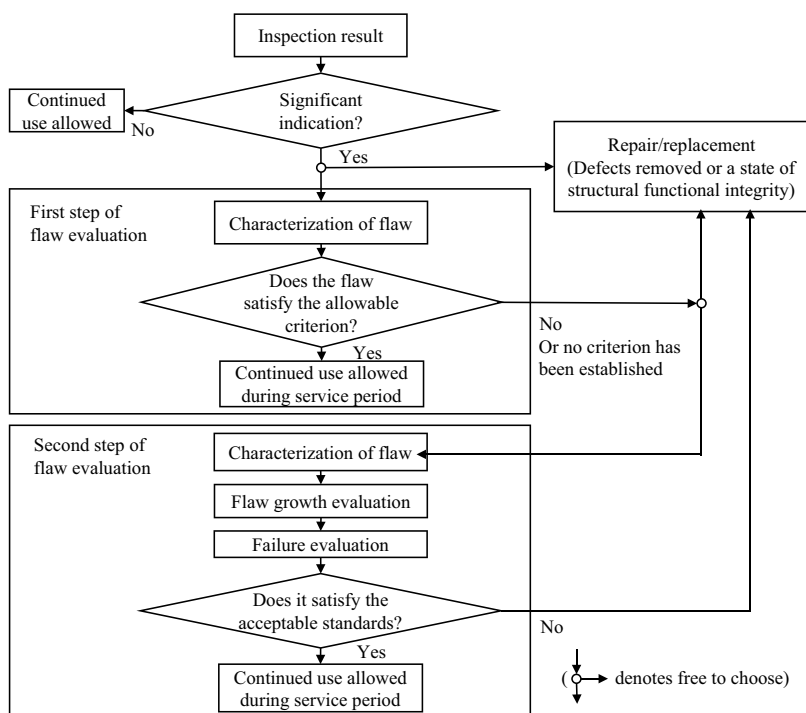


Fig. 11.4 Flow of integrity assessment in Fitness-for-Service Code (based on figure EB-1001-1 of JSME Fitness-for-Service Code)

Table 11.2 Examples of allowable planar flaw size for flaw in ferritic steel vessels. Depth of surface flaw a (in % or mm) against thickness t (mm)

Aspect ratio a/ℓ	Thickness $100 \leq t < 300$	Thickness $300 \leq t$
	a (%)	a (mm)
0.50	5.2	15.6
0.45	5.1	15.3
0.40	5.0	15.0
0.35	4.4	13.2
0.30	3.8	11.4
0.25	3.3	9.9
0.20	2.8	8.4
0.15	2.5	7.5
0.10	2.2	6.6
0.06	2.0	6.0

Note ℓ denotes the flaw length
(Based on Table EB-2001-1 of 2016 edition of JSME Fitness-for-Service Code)

location, dimension, and others. The size of a characterized flaw is compared with the size of the allowable flaw. When it is smaller than the allowable flaw, the component in question is judged as being allowed for continued use. When it is larger than the allowable flaw, the second step of flaw evaluation shown in paragraph (2) below is conducted.

The “size of the allowable flaw” mentioned above refers to small flaws that will not lead to failure as a result of the flaw growth during the lifetime of the component in question. Table 11.2 shows examples of allowable flaw size in the case of planar flaws on ferritic steel vessels.

(2) Second step of flaw evaluation (flaw evaluation considering flaw growth during the in-service period)

With an evaluation period specified for the characterized flaw size, a flaw growth analysis is conducted considering fatigue or stress corrosion cracking (SCC). This analysis enables the flaw size at the end-of-evaluation-period to be predicted. The acceptable flaw sizes for the Service Levels A and B, and the Service Levels C and D, determined by using the fracture evaluation method specified for each of the components in service, are used as the acceptance criteria. When the predicted flaw size satisfies the criteria, it is allowed to continue using the component, and the allowed using period is the evaluation period. A component that doesn’t satisfy the acceptance criteria is put to another evaluation period, or it will be repaired or replaced.

The techniques of flaw growth analysis are shown in Sect. 11.5.3 for cyclic load and in Sect. 11.5.4 for constant load (for SCC).

In terms of failure evaluation techniques, the fracture mechanics evaluation method based on fracture toughness and stress intensity factor is applied to ferritic

steel vessels, whereas the limit load evaluation method (equivalent to the net section stress collapse criteria mentioned in Chap. 10), the elastic–plastic fracture mechanics evaluation method, or the two-parameter evaluation method are applicable to austenitic stainless steel pipes. The details of these techniques are omitted here.

Taking ferritic steel vessels as examples, the acceptance criteria at the end-of-evaluation-period are shown below:

(1) Acceptance criteria for flaw size.

(a) Acceptance criteria for normal and upset conditions

$$a_f < 0.1a_c \quad (11.12)$$

where a_f denotes the predicted depth of a planar flaw and a_c denotes the acceptable depth of a flaw for normal and upset conditions.

(b) Acceptance criteria for emergency and faulted conditions

$$a_f < 0.5a_i \quad (11.13)$$

where a_f denotes the predicted depth of a planar flaw and a_i denotes the minimum value of the acceptable flaw depth for emergency and faulted conditions.

(2) Acceptance criteria based on the stress intensity factor

(a) Acceptance criteria for normal and upset conditions

$$K < K_{Ia} / \sqrt{10} \quad (11.14)$$

(b) Acceptance criteria for emergency and faulted conditions

$$K < K_{Ic} / \sqrt{2} \quad (11.15)$$

[2] Contents of the JSME fitness-for-service code

The Evaluation Section of the Fitness-for-Service Code consists of the main body and appendixes, where the main body is further divided into the general items for evaluation and the flaw evaluation for specific components. The “general items for evaluation” do not specify technical matters but give general provisions such as the definition of evaluation and the handling of criteria. The flaw evaluation for specific components shows the basics for flaw evaluation. The allowable flaw sizes are given for Class 1 components, and the flaw evaluation is specified for ferritic steel vessels, austenitic stainless steel pipes, and ferritic steel pipes.

There are 17 appendixes. The appendixes describe the procedure of flaw characterization, the calculation method of crack growth rates, the evaluation method of flaw shapes, the calculation method of stress intensity factors, the determination method of fracture toughness, the load conditions used in the flaw evaluation, the method of evaluating fractures (limit load evaluation method, elastic–plastic fracture mechanics evaluation method, and two-parameter evaluation method). Flaw evaluation methods for different core internals are also described.

In addition, the Fitness-for-Service Code includes an “interpretation” which describes the concept behind the code and technical basis for the content of individual rules.

11.5.3 Flaw Growth Evaluation (for Cyclic Load)

In general, the flaw growth under cyclic load can be expressed by the Paris’ law. In other words, the crack growth rate can be given by the following expression (identical to Eq. (10.22) in Sect. 10.10) using the range of the applied stress intensity factor, ΔK , between the maximum and minimum values of the stress intensity factor K , which are determined corresponding to the maximum and minimum values of stress in a load cycle.

$$\frac{da}{dN} = C(\Delta K)^m \quad (11.16)$$

where a denotes the flaw size, N denotes the number of load cycles, and C and m are parameters dependent on the material and environmental conditions.

A summary of the flaw growth evaluation is listed below:

- (1) Determine the stress distribution applied on the flaw surface using FEM analysis or other method after the flaw characterization.
- (2) Determine the stress intensity factor K at the flaw tip (corresponding to flaw length a).
- (3) Obtain the range of the applied stress intensity factor $\Delta K (= K_{\max} - K_{\min})$ from the range of stress.
- (4) Calculate the amount of flaw growth per stress cycle $\Delta a (\Delta a = C\Delta K_m)$.
- (5) Update the flaw length ($a = a + \Delta a$).
- (6) Repeat Steps (2) to (5) until the end of stress cycle.

For the same ΔK , the larger the stress ratio $R (= K_{\min}/K_{\max})$ value is, the faster the crack grows. In other words, C in the Eq. (11.16) is a function of R .

The crack growth rate in the environment is higher than that in the air atmosphere in some cases, and the involvement of the effects of the water environment, stress ratio, flow velocity, cyclic rate, and other effects makes the determination of the crack growth rate complicated. Therefore, researches based on the mechanism of flaw growth and the environmental effect are still underway.

11.5.4 *Flaw Growth Evaluation (for Time-Dependent Flaw Growth: SCC)*

Regarding stress corrosion cracking (SCC), which exhibits a time-dependent growing characteristic, the crack growth rate (da/dt) is determined by the stress intensity factor K under a given load as well as the materials and environmental conditions, as shown in the following equation (identical to Eq. (10.23) in Sect. 10.9).

$$\frac{da}{dt} = C'K^n \quad (11.17)$$

where a denotes the flaw size, and C' and n are constants depending on the environment and materials.

A summary of the flaw growth evaluation is listed below:

- (1) Determine the stress distribution applied on the flaw surface using FEM analysis or other methods after the flaw characterization.
- (2) Determine the stress intensity factor K at the flaw tip in question (corresponding to flaw length a).
- (3) Calculate the amount of crack growth Δa ($\Delta a = C K^n \Delta t$) from the increment of the stress holding time (Δt).
- (4) Update the flaw length ($a = a + \Delta a$).
- (5) Repeat Steps (2) to (4) until reaching the holding time to be evaluated.

The JSME Fitness-for-Service Code defines crack growth rate equations for austenitic stainless steels as shown in Table 11.3. A number of studies are underway on the mechanism of stress corrosion cracking, to determine with the constants C' and n in Eq. (11.17) on the basis of findings on the effects of the water environment (such as the dissolved oxygen concentration).

11.6 Examples of Integrity Assessment During the Service Lifetime

11.6.1 *Pressurized Thermal Shock (PTS)*

This event occurs when the inner wall of a reactor pressure vessel under pressure is cooled down abruptly which leads to tensile stress due to thermal stress. This event is peculiar to pressurized water reactor (PWR).

When a pipe rupture accident occurs in the reactor coolant piping system and the pressure in the reactor pressure vessel starts to decrease as a result, the emergency core cooling system (ECCS) supplies water to the core (which corresponds to the Service Levels C and D).

Table 11.3 Equations to evaluate the SCC crack growth rate of austenitic stainless steel materials

(a) Austenitic stainless steels for piping		(in MPa√m for K and m/s for da/dt)	
Environmental condition	SCC growth rate equation		
	Sensitized SUS304		
In the environment of normal reactor coolant water chemistry Electrochemical potential(ECP): - 70 mV _{SHE} ≤ ECP Conductivity < 20 μS/m	(i) 0 < K < 4.9 da/dt = 2.0 × 10 ⁻¹² (ii) 4.9 ≤ K ≤ 57.9 da/dt = 6.31 × 10 ⁻¹⁴ K ^{2.161} (iii) 57.9 < K da/dt = 4.1 × 10 ⁻¹⁰		

(b) Austenitic stainless steels for piping and core internals

Environmental condition	SCC growth rate equation	
	Sensitized SUS304	Low-carbon stainless steels
In the environment of normal BWR reactor water chemistry Conductivity < 20 μS/m Electrochemical potential (ECP) ≥ 150 mV _{SHE}	(i) 0 < K < 3.4 da/dt = 2.0 × 10 ⁻¹² (ii) 3.4 ≤ K ≤ 57.9 da/dt = 1.43 × 10 ⁻¹³ K ^{2.161} (iii) 57.9 < K da/dt = 9.2 × 10 ⁻¹⁰	(i) 0 < K < 6.7 da/dt = 2.0 × 10 ⁻¹² (ii) 6.7 ≤ K ≤ 57.9 da/dt = 3.33 × 10 ⁻¹⁴ K ^{2.161} (iii) 57.9 < K da/dt = 2.1 × 10 ⁻¹⁰
In the environment of BWR hydrogen water chemistry Conductivity < 20 μS/m Electrochemical potential (ECP) ≤ -100 mV _{SHE}	(i) 0 < K < 12.9 da/dt = 2.0 × 10 ⁻¹² (ii) 12.9 ≤ K ≤ 57.9 da/dt = 7.94 × 10 ⁻¹⁵ K ^{2.161} (iii) 57.9 < K da/dt = 5.1 × 10 ⁻¹¹	(i) 0 < K < 19.3 da/dt = 2.0 × 10 ⁻¹² (ii) 19.3 ≤ K ≤ 57.9 da/dt = 3.33 × 10 ⁻¹⁵ K ^{2.161} (iii) 57.9 < K da/dt = 2.1 × 10 ⁻¹¹

(c) Austenitic stainless steels for core internals

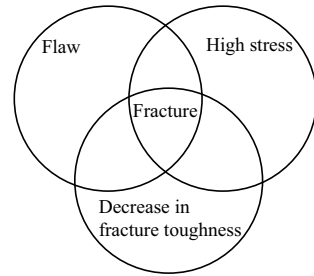
Environmental condition	SCC growth rate equation	
	SUS304 and low-carbon stainless steels	
High irradiation fluence domain (Neutron fluence > 5 × 10 ²⁴ n/m ²) In the environment of BWR reactor water	da/dt = 7.8 × 10 ⁻⁵ n ^{3.6} (CK ⁴) ⁿ n: Constants for the change in current density at the flaw tip as a function of corrosion potential, conductivity, sensitization, and neutron fluence (given in tabular form) C: Correction factor for flaw tip strain rate (i) 5.0 × 10 ⁻²⁴ n/m ² < neutron fluence ≤ 3 × 10 ²⁵ n/m ² C = 1.14 × 10 ¹³ ln (neutron fluence) - 6.03 × 10 ¹² (ii) 3 × 10 ²⁵ n/m ² < neutron fluence C = 6.59 × 10 ¹³	

In the above equations, da/dt = 0 when K ≤ 0
(Based on SCC crack growth rate diagram of 2016 edition of JSME Fitness-for-Service Code)

This cools down the inner wall rapidly with the pressure acting on the wall. The rapid cooling produces tensile stress on the inner surface of the vessel due to thermal stress. If a flaw exists on the locations like the inner surface, and the fracture toughness of the material is reduced due to the neutron irradiation during operation, the high stress caused by the PTS event may lead to non-ductile fracture (Fig. 11.5).

It is mandatory to predict the fracture toughness of the materials during its use at the design stage and to check it by means of the surveillance test conducted periodically during the service lifetime. The prediction and checking are done because it is known that the ferritic steel material forming a reactor pressure vessel, irradiated with neutrons from the core during operation, suffers a reduction in the fracture toughness.

Fig. 11.5 Condition for fracture occurrence due to PTS events



In addition, inspection by ultrasonic testing is performed during the service lifetime to make sure that no flaw exists.

Moreover, the assessment based on fracture mechanics is conducted to prevent the PTS event from causing brittle fracture. In other words, the stress intensity factor is calculated on the assumption that a flaw exists at the time of occurrence of a PTS event, and the occurrence of fracture is assessed by making comparison with the fracture toughness value at the time of assessment estimated by the result of surveillance test.

Assessment method for PTS events is stipulated in JEAC4206 “Method of Confirmation Tests of the Fracture Toughness for Nuclear Power Plant Components” in the Nuclear Power Section of Japan Electric Association Codes.

11.6.2 Integrity Assessment Method of a Reactor Pressure Vessel in the Service Levels C and D

This section describes the overview of the integrity assessment method on the basis of JEAC4206-2007, Appendix C “Assessment Method for Preventing Non-ductile Fracture in the Core Region of a Pressurized Water Reactor Pressure Vessel in the Service Levels C and D.” Fig. 11.6 shows the flow of this integrity assessment.

(1) Establishing a PTS state transition curve

- Conduct thermal hydraulic analysis of the reactor coolant temperature, pressure, and flow rate for the PTS events to be assessed (main steam line break accident, small-break loss of coolant accident, and large-break loss of coolant accident).
- Assess mixing of coolant for the case of loop flow stoppage.
- Determine the transients in the reactor coolant temperature, pressure, and flow rate near the core cooling zone.
- Conduct stress analysis in the vessel wall of core region.
- Determine the heat transfer coefficient in the coexisting state of natural and forced convection using the Jackson-Fewster equation.

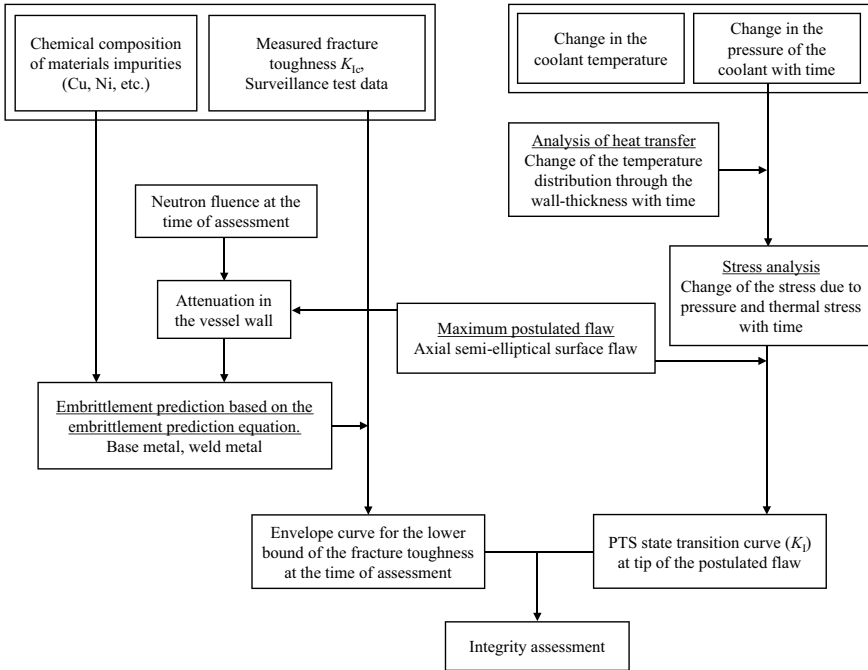


Fig. 11.6 Flow of integrity assessment considering PTS events (based on Annex Figure C-3320-1 in JEAC4206-2007)

- Calculate the stress intensity factor at flaw tip for a postulated flaw on the basis of the stress distribution.

(2) **Establishing a fracture toughness transition curve**

- Estimate the amount of neutron fluence at the flaw tip with the attenuation inside the plate thickness taken into consideration.
- Shift the fracture toughness data in the core region material under consideration in parallel with the temperature axis. To envelope the lower bound, giving the T_p in the following equation can set the fracture toughness transition curve.

$$K_{IC} = 20.16 + 129.9 \cdot \exp[0.0161(T - T_p)] \quad (11.18)$$

If the measured values from the surveillance tests are available, the amount of the shift of fracture toughness data (ΔK_{IC}) will be corrected to the value corresponding to the assessment time by the domestic embrittlement prediction equation, and the fracture toughness transition curve will be set to envelope the lower bound of fracture toughness data. If the measured value from the surveillance test is not available, the fracture toughness transition curve

will be set by the K_{IC} curve that is typical for such materials and by ΔRT_{NDT} corresponding to the assessment time which is determined by the domestic embrittlement prediction equation.

(3) Integrity assessment method

- Maximum postulated flaw: An axial semi-elliptical surface flaw with a depth of 10 mm and a length of 60 mm is assumed on the inner surface of the core region.
- Integrity assessment: The PTS state transition curve at flaw tip of the assumed flaw is compared with the fracture toughness transition curve that takes irradiation embrittlement at the time of assessment into account. If they do not cross each other, the pressure vessel is assessed as having integrity.

11.6.3 Integrity Assessment Method of a Reactor Pressure Vessel in the Service Levels C and D (Edition of 2016)

As mentioned in Sect. 11.4, a major revision on JEAC 4206 was conducted in 2016, where the integrity assessment on Service Levels C and D are also comprehensively revised to reflect the latest findings. The flow of integrity assessment in JEAC 4206-2016 is shown in Fig. 11.7. The main revisions from edition 2007 are described as follows.

(1) Establishing a PTS state transition curve

- The maximum postulated flaw is set as an under-clad semi-elliptical flaw, and effects from factors such as clad and welding residual stress are considered in calculating the stress intensity factors.
- Based on the results of nondestructive inspection, it is allowed to assume a smaller flaw.
- The evaluation equation for the stress intensity factor for the under-clad flaw is stipulated.
- It is allowed to calculate the stress intensity factors at the deepest point of the maximum postulated flaw by FEM analysis.

(2) Establishing a fracture toughness transition curve

- Instead of the conventional method that empirically estimates the lower bound, by referring to the reference temperature (T_0) in the master curve method, the fracture toughness transition curve is set based on the method to determine the confidence limit considering the inherent characteristics in statistical distribution of fracture toughness such as the uncertainties in embrittlement prediction and the uncertainties in evaluating T_0 .
- By referring to the Charpy transition temperature (T_{r30}), and by considering the uncertainties in embrittlement prediction and the correction value dependent on the steel types, also by referring to the concept behind the master curve

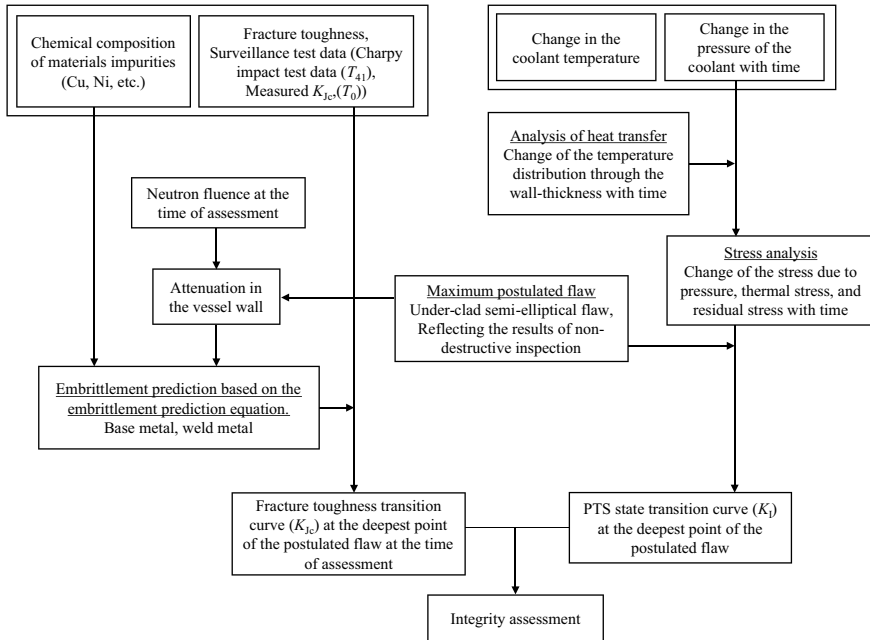


Fig. 11.7 Evaluation flow for PTS events in JEAC4206-2016

method, the fracture toughness transition curve is set based on the method to determine the confidence limit considering the inherent characteristics in statistical distribution of fracture toughness.

(3) Integrity assessment method

- In the general assessment, the occurrence of non-ductile fracture is assessed by considering the large break lost-of-coolant accident (LBLOCA) which is the most severe accident in temperature transient in evaluation. If the criteria of general evaluation are not satisfied, detailed evaluation will be conducted.
- In the detailed assessment, the LBLOCA, the small break lost-of-coolant accident (SBLOCA), and the main steam line break accident (MSLB) are considered to assess the occurrence of non-ductile fracture that considers warm pre-stress (WPS) effect, and also to assess the arrest of flaw propagation.
- If the criteria of detailed assessment are not satisfied, the following corrective actions will be taken, after which the general assessment or the detailed assessment will be conducted again.
 - Mitigating the decrease in fracture toughness by reducing the neutron fluence received by RPV.
 - Recovering the fracture toughness of RPV by annealing.

- Mitigating the temperature decreasing and stress increasing in the core region of PRV by increasing the temperature and decreasing the flow rate of safety injection water.

11.7 Leak Before Break (LBB)

When crack grows and propagates due to corrosion or fatigue in components and pipes, it may not lead to an instantaneous occurrence of large-scale break but instead it may penetrate through the wall and thereby cause the coolant inside to leak outward visibly or detectably. Such phenomenon is called leak before break (LBB).

In piping systems in the nuclear power plant, full circumferential break of piping is postulated to secure the integrity of components surrounding the pipe when this postulated accident occurs, and protective facilities such as pipe whip restraints are installed at various locations. In the nuclear power plant, the leak of reactor coolant from the pressure boundary is monitored during operation. Consequently, if enough time is assured from the occurrence of a leak, measures such as shutdown of the reactor can be taken so as to prevent the break of the pressure boundary. For this reason, if LBB is valid, these protective facilities become unnecessary, which brings about advantages of reduction in facility expenses and reduction in exposure dose arising from inspection.

Demonstrative tests on LBB have been carried on, and the Technical Guidelines for Protection Design against Postulated Piping Failures in Nuclear Power Plants (JEAG 4613-1998) including the assessment of LBB justification for austenitic stainless steel pipes are established. In March 2003, the Japan Society of Mechanical Engineers issued the Code for Nuclear Power Generation Facilities—Rules on Protection Design against Postulated Pipe Rupture for Nuclear Power Plants [4] on the basis of the JEAG 4613 Guideline. This code reflects up-to-date results of research on pipes made of austenitic stainless steels, carbon steels, and low-alloy steels used in piping systems in the reactor coolant pressure boundary and in the main feed water and the main steam piping systems in PWR containment vessels.

Figure 11.8 shows the concept of LBB. The illustration shows how a circumferential small flaw on the inner surface of a pipe grows with time. In stage (ii), the crack grows from the inner surface to the outer surface, but since it does not reach the outer surface, it is still a non-penetrated flaw. When the flaw reaches stage (iii), the front tip of the flaw reaches the outer surface of the pipe, so that the fluid inside starts leaking. As more time passes, the crack grows remarkably even on the outer surface. This state is called a penetrated flaw (stage (iv)). As the leak from the penetrated flaw increases significantly and the amount of leak per unit time (namely the leak rate) exceeds a certain level, the leak detection system is actuated and the occurrence of the leak is detected.

As the crack propagation causes the load bearing capacity to decrease, unstable fracture may occur at a certain stage. The judgment on the occurrence of LBB can be made by making it clear that at which stage of crack propagation such unstable

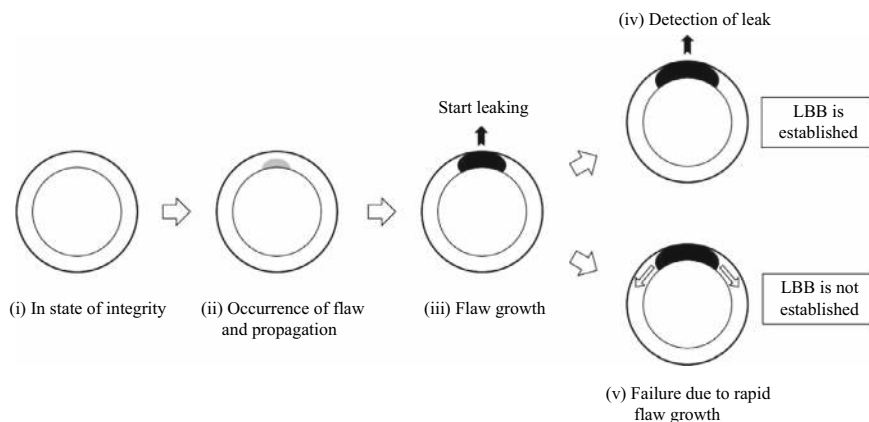


Fig. 11.8 Concept of LBB for piping

fracture occurs. For example, if a crack rapidly propagates in stage (ii) to (iii) in Fig. 11.8 and unstable fracture occurs as stage (v), the concept of LBB will be not established.

As described above, the Japan Society of Mechanical Engineers: Rules on Protection Design against Postulated Pipe Rupture for Nuclear Power Plants (JSME S ND1 2002) stipulates rules for the assessment of pipe failure modes and crack opening areas based on the LBB concept.

Here, “protection design” means an assessment of strength after considering the postulated location of pipe rupture, the mode and crack opening area of the rupture, determining events postulated at the time of pipe rupture, and implementing protective measures.

The steps for justifying the occurrence of LBB are shown, for example, in Figure D-2100-1 of JSME S ND1 2002.

Here, the preconditions for justifying the occurrence of LBB have an operational management aspect and a structural integrity aspect. The former aspect includes the capability to detect a leak of 1 gpm ($0.23 \text{ m}^3/\text{h}$) within an hour and the preparedness to shut down the reactor as soon as possible if the leak from the main steam piping or main feed water piping exceeds this value. The latter aspect includes compliance with structural technical standards and preventive measures against damage due to SCC and erosion corrosion.

In defining a postulated flaw in the steps of assessing the justification of LBB occurrence, a five-times margin (namely, 5 gpm as the amount of leak rate for the postulated flaw) is set for the leak detection facility. The primary stress and the thermal expansion bending moment are considered as the applied load in consideration of the Service Levels A, B, and C as well as the combination of the Service Level A and the earthquake.

Whether the event is a leak or a rupture is determined through comparison between the applied stress and the stable limit stress, and in the case of a leak, the crack opening area is determined by the Tada & Paris method to calculate the jet force.

Exercises

1. The equation of crack growth rate due to cyclic stress is commonly called the Paris' law, being defined by the range of the applied stress intensity factor ΔK [MPa $\sqrt{m}$] between the maximum and minimum of the stress intensity factor K determined by the maximum and minimum of the stress. Show this equation of crack growth rate. Also show the outline of the crack growth rate diagram based on the Paris' law.
2. Describe the outline of the evaluation method of flaw growth due to stress corrosion cracking (SCC). Show the outline of the SCC growth rate diagram that is used to assess SCC growth in austenitic stainless steels (Type304). Define the vertical and horizontal axes in the diagram clearly.
3. Briefly describe the integrity assessment system to address flaws detected in inspection, mainly from the viewpoint of the difference between the two steps of the assessment.
4. Briefly describe the purpose of the surveillance test. Enumerate two types of specimens placed as surveillance specimens inside a reactor pressure vessel. Also show what each of these specimens is used in assessment.

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