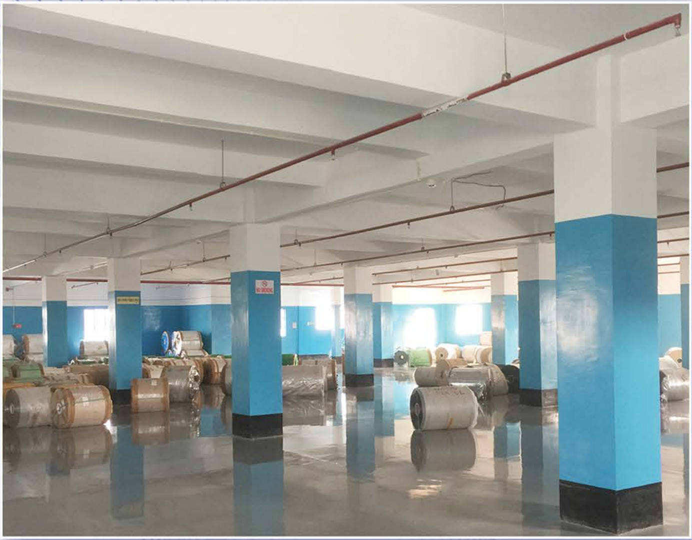
REINFORCED CEMENT CONCRETE STRUCTURES

PRACTICAL DESIGN AND STANDARD DETAILS



ASHOKE KUMAR DASGUPTA

CRC Press
Taylor & Francis Group

Reinforced Cement Concrete Structures

This book aims to provide actual methods of calculation and standard details followed by professionals in industrial projects pertaining to Reinforced Cement Concrete (RCC) structures backed by practical design and standard details. It covers the engineering properties of soil and types of tests, different types of concrete grades, standard notes and codes, and workout examples of piles, foundations and superstructure elements. It provides all of the standard construction details, including reinforcement arrangements, generally used for RCC works in superstructures and foundations.

Features:

- Provides the strength design calculation for foundation and settlement analysis of the founding soil together.
- Discusses standard details of reinforced concrete joints and reinforcement placement.
- Describes suitable types of material and selection of structure according to the nature of the founding soil and service life of the plant.
- Explores standard construction details.
- Includes solved problems, design and workout examples as per Indian and US standards.

This book is aimed at professionals in construction, structural and civil engineering.

Reinforced Cement Concrete Structures

Practical Design and Standard Details

Ashoke Kumar Dasgupta

CRC Press
Taylor & Francis Group
Boca Raton London New York

CRC Press is an imprint of the
Taylor & Francis Group, an **informa** business

Designed cover image: © Ashoke Kumar Dasgupta

First edition published 2025
by CRC Press
2385 NW Executive Center Drive, Suite 320, Boca Raton FL 33431

and by CRC Press
4 Park Square, Milton Park, Abingdon, Oxon, OX14 4RN

CRC Press is an imprint of Taylor & Francis Group, LLC

ISBN: 978-1-032-53878-5 (hbk)
ISBN: 978-1-041-02199-5 (pbk)
ISBN: 978-1-003-61811-9 (ebk)

DOI: 10.1201/9781003618119

Typeset in Times
by Apex CoVantage, LLC

To my parents

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Contents

Preface

This book presents a guideline that systematically bridges the gap between university curriculum and practical work in the field. It is intended for fresh graduate engineers, associate engineers, design and construction engineers and working professionals in civil engineering.

The focus is kept on the practical design and construction detailing of Reinforced Cement Concrete (RCC) structures, including foundation systems for industrial plants.

This book contains workout examples providing the design of items commonly used in RCC structures. These examples are straightforward and based on professional experience that will inspire engineers to design structures on their own or solve challenges in construction sites without the help of expensive design software. American standards ACI, ASCE and Indian Standard Codes have been used in workout examples.

Numerous examples of construction details are provided here with illustrative sketches, which are not available in comparable textbooks. This book will also assist engineers in evaluating the outputs and drawings generated using computational methods of designing software used in project design offices.

Acknowledgments

To my colleagues, family and publisher who encouraged me to write this book. Special thanks to the team at Taylor & Francis, Mr. Sukdeb Sau, my colleague for preparing drawings, Mr. Subhendu Baran Roy, my brother-in-law, Dr. Suman Dasgupta, my son for suggesting improvements to the manuscript and members of the Dasgupta family for constant inspiration and support.

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Ashoke Kumar Dasgupta has worked in reputed consulting engineering companies, Development Consultants Pvt Ltd (India) and The Kuljian Corporation (USA), at their design offices and construction sites in India, the USA, Bangladesh, Middle Eastern countries (Turkey, Saudi Arabia, Syria, Egypt, the UAE, Qatar, Kuwait), Far East countries (China, Japan, Indonesia) and Vietnam from 1974 to 2019. His expertise lies in structural, civil and foundation engineering for power plants and industrial structures.

This book is his third publication with Taylor & Francis. His previously published books are *Design of Industrial Structures: Reinforced Cement Concrete and Steel* (2022) and *Design of Structural Steel Joints* (2024).

1 Introduction

1.1 INTRODUCTION

The *need for learning basic element design* is essential for a designer to build up his confidence level while entering the intriguing design steps of Reinforced Cement Concrete (RCC) design. The analysis and design software used in the preparation of RCC structures needs member input information, e.g., dimensional sizing of frame elements, such as depth and width of columns and beams, to prepare the analytical model of any building or structure. So, the designer should have a proven knowledge while selecting member sizes or making preliminary designs of members to proceed with the frame analysis and subsequent reinforcement design. The knowledge of basic element design also helps an engineer to find out errors, if any, in output results from software design and construction drawings. The engineers working in the construction field should have knowledge in the basic design of elements to understand the stresses in members and select the appropriate method of formwork and dismantling, placement of reinforcement and splice joints, sequence of casting concrete and monitor the quality control during construction. Knowledge in basic design enables construction engineers working at remote sites to take care of emergency situations.

The basic element design is to calculate the design strength of members and their joints in terms of moment, shear, and torsion, axial and bearing strength. The design strength is multiplied by a strength reduction factor to get the nominal or allowable strength of a member. The nominal strength shall be greater than or equal to the required strength of members, which is calculated for the factored load and combinations according to the applicable RCC design codes and standards.

1.1.1 Type of loads and combinations

The designer should select the *type of loads and combinations* with appropriate load factors so that the structure and elements can withstand all the loads safely, maintaining durability requirements throughout the plant design life cycle.

The following codes and standards are used for the selection of loads and load combinations to be applied on buildings and structures for analysis and member design:

- American standards – ASCE 7, ACI 318
- Indian standard codes – IS: 875, IS: 456, IS: 1893

The types of loads generally used for common buildings and structures are given below for the user's ready reference.

DOI: 10.1201/9781003618119-1

1.1.1.1 Dead loads (DL)

Reinforced concrete	24 kN/cum
Plain concrete	23 kN/cum
Floor finish, plaster	20 kN/cum
Roof treatment with tile	1.55 kN/sqm
Brick masonry	22 kN/cum
Soil	19 kN/cum

1.1.1.2 Live loads (LL)

This live load means superimposed loading on floors and roofs that are movable. Minor equipment loads may also be included within this category. The equipment load is the total weight divided by its base area. Intensity of live loads in general buildings is furnished in relevant codes and practices.

1.1.1.3 Wind load (WL)

Intensity of load and its method of computations are available in standard codes and practices. Design wind pressure at different heights should be calculated and presented in the design basis criteria or design document.

1.1.1.4 Seismic load (SL)

Seismic design criteria for the design and construction of buildings and structures subject to ground motion are presented in relevant codes and standards. The seismic coefficient (response spectra/static force method using coefficient) should be as stated in applicable codes.

1.1.1.5 Temperature load (TL)

Temperature load results from thermal stress of framing members due to environmental temperature fluctuations. Since framing members are self-restrained, free thermal expansion or contraction is not possible, resulting in these thermal stresses. The design range should be the average of maximum and minimum operating temperatures. For example, if the highest temperature is 55°C and the lowest temperature is (–) 5°C, the thermal range for strength design purposes should not be less than (+ / –) 30°C, unless otherwise specified in contract specifications.

In addition, various loads should be considered, such as earth pressure on earth-retaining structures, hydrostatic load on water-retaining structures, blast loads on special blast-resisting walls of buildings, impact loads, snow load, ice load and rain loads for site-specific and special structures.

1.1.1.6 Combination of loads

Designers should consider all the possible combinations to get the worst developed stress on members. A few typical cases of combination of loads are provided below for guidance.

1. DL + LL + LLR
2. DL + WL
3. DL + SL

4. DL + LL +WL
5. DL + % LL +SL
6. DL + LL + TL
7. DL + WL + TL
8. DL + % LL +SL + TL
9. DL + LL + 50% WL

The reversal of transient loads (-WL, -SL, -TL) shall be considered in addition to the above load combinations.

If there is crane load (CL) in the building, the number of combination cases shall be increased with applicable crane load cases, including horizontal surge. For crane combinations, DL + CL + 50% WL is found critical for some buildings.

DL = dead load
LLR = roof live load
LL = super imposed load or live load on floors
WL = wind load
SL = seismic load
TL = environmental temperature load
CR = crane load

1.1.2 Method of design

RCC structures are normally designed by the *limit state method (load factor method).* The acceptable limit for the safety and serviceability requirement before failure is called a limit state. It should also satisfy the *limit state of serviceability* requirements, such as the allowable limit of deflection or cracking. Where the limit state method cannot be adopted, the structure should be designed by the *working stress* method.

There are three *methods of strength design* – Working stress method, Limit state method and Ultimate load method.

The Working stress method is old and simple in concept. It follows the elastic theory of reinforced concrete following Hooke's law (*the strain of material is proportional to applied stress within the elastic limit of the material*). In this method, tensile stress is taken by steel only. Strain in steel is equal to strain in concrete. The compressive stress in steel is equal to compressive stress in concrete multiplied by a factor called the modular ratio of the concrete. Modular ratio, m = Es/Ec (the ratio of the modulus of elasticity of steel to that of concrete). But this value varies with the grade of concrete mix. So, it is considered as $m = 280 / \sigma cbc$, where σcbc is the permissible stress in bending compression for different grades of concrete. The permissible stress in steel and concrete shall not exceed the value obtained by dividing the characteristic strength of the material by the factor of safety (3 for concrete and 1.8 for steel). The working stress method gives a safe design with less deflection or cracks, but the section sizes are higher than other methods of design.

The Ultimate load method considers ultimate strengths at ultimate load.

The Limit state method is currently chosen as the best method in strength design. It allows design load up to its characteristic load multiplied by the *partial safety* *factor appropriate to the nature of loading and also the limit state that is* being considered. The design strength of the material is the characteristic strength of the material divided by the *partial safety factor appropriate to the material* (1.5 for concrete and 1.15 for steel) *and also the limit state* that is being considered.

There are two limit state concepts – one is the limit state of collapse and another is the limit state of serviceability.

The limit state of collapse of a structure could be assessed from rupture or buckling due to elastic or plastic instability. At this limit state, the structure will undergo collapse or become unstable under any combination of expected overloads.

The limit state of serviceability ensures that the structure will not exceed the permissible limit of deflection and that crack width shall be within acceptable limits for all combinations of loads (Dead Load, Live load, Wind load/Seismic load and others) multiplied by the partial safety factor of the limit state of serviceability.

Workout examples of member strength designs are presented in Chapter 5.

1.1.3 Selection of materials for structures

The selection of materials for the structure should be carefully decided by the designer. The reinforced concrete structure is less costly than a steel structure and needs low maintenance; it has become a popular choice for a long time. The reinforced cement concrete (RCC) structure is used to build buildings, roads and bridges, crossing works, flood protection barriers, seawalls and shore protection work, dams and river training works, canal lining, electrical poles, rail sleepers, high-rise chimneys, industrial structures and more.

RCC structures are functionally efficient and maintenance-free. High-strength concrete is used to make the member sizes slim and thus to provide more clear space on floors. Precast concrete elements are often used in the building industry to save *time in construction* and make the project cost-efficient. Advance construction and design methods are used to manufacture high-strength precast pre-stressed concrete elements for pile foundations and building elements for multi-story buildings, car parks, workshops and storage sheds. The heavy-depth bridge girders are now replaced by lightweight and slim pre-stressed concrete members.

The cement concrete structure does not require periodic maintenance like the painting of a steel structure. The use of sulfate-resistant cement, Portland pozzolana cement and mineral additions like ground granulated blast furnace slag and epoxy-coated bars/corrosion-resistant bars enhances the durability and life of concrete structures in extreme environments, including marine conditions.

For large projects, the quality of materials and workmanship is monitored by qualified engineers. Modern equipment like batching plants, transportation trucks fitted with rolling mixers, concrete pumps and multi-facility cranes, etc., are used for construction.

Concrete structure is not a homogeneous product like steel sections. This is a cast-in product that works in combination with its reinforcement steel bars and cement concrete mix hardened to its characteristic strength. The hardened concrete bears compression and shearing forces; flexural stress is taken by reinforcement steel bars. Shearing force in excess of concrete capacity is shared by reinforcement bars in the form of stirrups and diagonal bars. Thus, the cost of concrete is dependent on the ratio of unit prices of steel bars and the grade of concrete mix. Higher grade means higher strength, which generally requires more cement consumption. Hence, the economy in overall cost can be achieved by balancing the quantity of reinforcement bars and the volume of concrete mix. High-strength concrete allows for a reduction in member sizes, which consumes less volume and also reduces the weight of the structure. It provides a direct saving on foundation costs for dead load predominant structures. However, one should remember that using high-strength concrete in construction requires perfect quality control.

Quality, cost control and time of construction are basic requisites that should be considered while building any structure. Hence, the use of factory-made ready-mix concrete, precast and pre-stressed materials has become the choice of the day for owners and constructors, including small residential house builders.

Materials are to be selected by the designer, considering overall economy and cost savings. Procurement of reinforcement steel bars of maximum available lengths can minimize loss due to wastage of steel.

The selection of member dimensions by the designer plays an important role in saving costs. Designers should use regular sizes of beams and columns that match the commonly available sizes of formwork materials; members of the same sizes allow multiple uses of formwork and save labour costs.

1.1.4 Soil and foundations

The soil and foundation system are an integral part of the building structure. Therefore, the foundation designer should be aware of the safe bearing capacity of the soil and the expected settlement of the foundation structure to ensure safety and performance during its service life. Chapter 2 provides a brief guideline about the soil and foundation system.

1.1.5 Drawings and construction supervision

Drawings for reinforced concrete structures are normally done in two parts – general arrangement or concrete outline drawing and reinforcement detail drawing. The concrete outline drawings are prepared as per architectural drawings. Dimensional details of the building structure in plan, section and elevations, number of floors, spacing of columns, arrangement of floor beams, height between floors, staircase and lift, member sizes, foundation outline, etc., all necessary to prepare formwork, are provided in the concrete outline drawing.

Reinforcement detail drawings show the arrangement, placement, numbers, sizes, and spacing of reinforcement steel bars in slabs, beams, columns, walls, stairs and foundations. Bar bending schedule drawings are also prepared for fabricating reinforcement bars at the site workshop or fabrication yard, when necessary.

For large buildings and industrial structures, separate drawings are prepared for deep excavation, formwork and scaffolding (*a temporary structure used to support workmen and materials*) as required for expediting construction work.

Construction supervision includes all the works starting from land survey, planning, engineering, procurement, supervision, payment and handing over the plant or building to the owner after maintenance over a specified period as per drawing, contract specification and approved drawing.

The engineering supervision or construction supervision is done to ensure that the construction work is in conformity with design drawings and technical specifications.

The work includes checking lines and layout, excavation, formwork, reinforcement work including bar bending schedule, embedded parts, concreting works, curing, quality control including testing of materials and concrete mix, certification of quantity, bills for payments, etc.

For large projects, there are quality control monitoring systems established by big construction companies based on their experiences. The construction engineer follows the same. Let us highlight some tests and procedures that are essential for the quality control work.

TABLE 1.1
Guidelines for testing construction materials and work.

SL No	Materials	Test to be done
1	Cement	As per relevant codal provision
2	Water	Impurity test (chloride ions, sulfate SO3, alkali carbonates and bicarbonates, other dissolved salts; total dissolved salts shall not be more than 3000 ppm)
3	Aggregates	Physical, mechanical and chemical properties; grading, moisture content, impurities
4	Admixture	As per ASTM C494 or local codes of practice
5	Form work	Material (waterproof, non-warping, non-shrinking); Shape, lines and dimensions, strength to support without deflection; adequacy of vertical supports, cross ties and bracings; surface finish and levels after completed
6	Reinforcing steel	Strength and material tests as per relevant code of practice; fabrication (shape, size and dimensions) and placement as per drawing/bar bending schedule; location and number of splices as per drawing
7	Concrete grade	Mix design at laboratory and full-scale batching plants; Testing of all properties at laboratory and full-scale batching plant trials (strength tests; durability tests; slump tests, tests for air content, temperature, density, initial and final setting times, bleeding)
8	Construction joints	Location and details of joints, quality and dimensions of water stops and splices, dimension and alignment
9	Anchor bolt and metal plate embedment	Material, paint or galvanization, sizes, lugs, levels and location dimensions
10	Placement of concrete	Placing condition (surface of formwork, preparation of old concrete joint, if any), weather condition, placing equipment and methods; placing time (time interval to avoid cold joints), rate of pouring, temperature monitoring, maximum free fall, vibrator uses, consolidation
11	Surface finishes	Repairing defects, screed or float finishes or by surface finishes tools for special type finish
12	Curing	Method and duration of time (a fully weight-curing system with damp hessian for minimum ten days following placement; use of curing compound shall be restricted to special situation)

For small job sites, where the availability of equipment, facilities, and skilled manpower is scarce, an engineering supervisor has to look into the following minimum requirements:

a. Excavation foundation pits: line and level; a plastic transparent water pipe filled with water may be used as a level tube.
b. Quantity of reinforcement and its placement.
c. Mix proportion of concrete. Where volumetric proportion is used, bulking of sand and grades of coarse aggregate should be checked. Coarse aggregates should be uniformly graded.
d. For hand-mixed concrete, the quantity of cement should be 10% extra.
e. Concrete mixing machines should be rotated for at least 2 minutes for each batch.
f. Water-to-cement ratio and slump should be controlled.
g. Time of placement should be within the initial setting time of cement (not more than 50 minutes from mixing water in cement to placement and consolidation).
h. Construction joints should be vertically faced and away from the column face.
i. Semi-hardened (cold concrete that has passed the initial setting time) should not be tamped to the correct level.
j. Use of vibrators should be restricted. Bars should be placed in multiple layers to avoid congestion at beam-column joints. Excessive use of vibrators touching re-bars, segregates aggregate and thus weakens the strength of concrete.

1.1.7 Design Guidelines

Design guidelines for RCC structures should be in line with standard engineering procedures followed in the design office. In general, the following steps are suggested for beginners and engineers as a standard work procedure

a. Study building architecture or equipment layout drawings
b. Review soil investigation reports and technical specifications
c. Collect civil input data
d. Prepare a design basis report or design criteria
e. Conceptual design and framework preparation
f. Load assessment
g. Analysis and strength design
h. Preparation of general arrangement (GA) or concrete outline drawing
i. Interdisciplinary checking of GA drawings
j. Incorporate change requirements in GA drawings
k. Prepare reinforcement drawings for construction
l. Submit drawings with supporting calculation for approval (where needed)
m. Release drawings for construction
n. Coordinate with the field for clarification of drawings, as and when required
o. As-built drawing

The above list of activities covers the majority of the designer's work. In addition, the design engineer also takes part in the preparation of the material procurement list, design coordination meetings, estimation of the bill of quantities, review of bar-bending fabrication drawings, etc. More details on how to prepare a Design Basis Report and conceptual design are available in my book *Design of Industrial Structure: Reinforced Cement Concrete and Steel.*

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2 Soil and foundation

2.1 INTRODUCTION

The building foundation is an integral part of its superstructure. So, the building designer should have a basic knowledge of soil and foundation engineering. The settlement behavior of founding soil and the safe bearing capacity are prime factors in choosing appropriate types of foundation systems as well as deciding column spacing for uniform load distribution on soil. Selection of the type of cement and reinforcement protection is based on sulfate and chloride content in groundwater and soil.

In this chapter, the reader will be given exposure to minimum requirements of soil and foundation systems.

2.2 ENGINEERING PROPERTIES OF SOIL

Following are the basic elements of soil that we can define to simplify the beginning of the discussion:

- Clay
- Silt
- Sand
- Rock

Clay (C) particles are microscopic and fine-grained. Clay is soft when mixed with water and hard in a dry state. It is cohesive in nature and has low permeability. It exhibits plasticity when mixed with water; for example, it does not show any cracks when rolled into a thread on your palm until the diameter reaches about 3 mm or less, but silt cracks.

Silt (M) is called rock flour. It is fine-grained soil without plasticity. If we mix silt with water and make a ball and shake it on the palm, the surface becomes glossy; however, the glossiness disappears when we tap a finger on it. It is not cohesive and will not stick to the palm like clay. The silt can be dusted off after the hand is dried.

Sand (S) is fine to coarse-grained cohesionless soil. The particles are yellowish in color and generally angular in shape. Yellow sand is used as fine-grain aggregate in cement concrete work and construction of roads. Coarse-grain sand is highly permeable. River sand is fine-grained and rounded in shape. White sand is a fine-grained material used for area filling work. Grain sizes larger than 4 to 5 mm are classified as gravel.

Gravels (G) are round-shaped and used as coarse aggregate for concrete work.

Rocks may be broadly classified into the following types:

DOI: 10.1201/9781003618119-2

a) Hard rock requiring blasting
b) Soft rock
c) Decomposed rock

Hard rocks are granite, quartz, sandstone, basalt and limestone. These are mainly used for coarse aggregates in cement concrete work and road construction after being crushed and graded into required sizes.

Soft rocks are marbles, slates, lateritic, coral and some kinds of limestone deposits.

Decomposed rocks are compacted natural aggregates of minerals, sand and rock particles that are bonded together.

Excavation of soft and decomposed rock can be done by jackhammers and mechanical excavators (by chiseling). It does not require blasting.

There are some other types of soils, like organic clay, black cotton soil, peats, etc., which are found to be highly compressible and possess swelling characteristics. These soils are identified as bad soil and are not suitable for bearing foundation load. If these types of soil are below the founding level, they have to be completely removed and replaced with good soil.

The above description states the properties of basic elementary soils. In real nature, soils are found in strata, i.e., several parallel layers of material arranged one on top of another. Each layer consists of naturally mixed-up soil like clayey sand (SC), i.e., about 50–60% sand with a fair clay binder; silty sand (SM) contains a very large portion of silt and about 10% gravel of 20 mm size. Sometimes, it can be seen in pure form, like layers of sand in different grain sizes mixed with gravels, or clay deposits of large depth or layers of silt without contamination.

Soil is a creation of nature; it is not like steel and concrete materials, which are manufactured by us. The behavior of concrete and steel materials can be ensured by their well-defined parameters like modulus of elasticity (tensile stress/tensile strain), yield stress and Poisson's ratio (transversal expansion/axial compression). But these parameters are not certain in the case of a soil structure formation on the surface of the Earth. The soil structure is neither isotropic nor homogeneous in nature. The properties of soil depend on its water content, porosity (percentage of voids), void ratio (ratio between the volume of voids and the volume of solid), permeability, loading pattern, i.e., history of formation, etc. For better understanding, let us consider the soil structure as a honeycombed structure made by its particles. The grains are adhered to each other by intermolecular attraction. This inter-granular force depends on molecular size as well as the surface of attraction. This force is predominant when the size of the particle is very small. There are pores in the honeycombed structure.

In clay, the grains are microscopic in size. Hence the inter-granular force is more compared to its molecular weight; as a result, the clay particles arrange themselves in flocs standing vertically or in any patterns parallel to each other depending on their sizes and power of inter-granular attraction. But in the case of sand, the inter-granular force is negligible because of its particle size. The molecules of sand roll down and occupy the spaces in between the honeycombed structure. Hence, the sand deposit is denser than that of clay.

A foundation engineer looks for good soil, i.e., the engineering properties of soil should behave consistently during the service life of the structure. It means the superstructure or foundation will not tilt or settle down beyond the permissible limit under the stipulated design load.

To ensure this requirement, the engineer has to determine the following engineering properties of the soil stratum:

- Description of soil
- Grain size distribution
- Density and moisture content
- Atterberg limits – liquid limits, plastic limit, plasticity index
- Safe bearing capacity
- Coefficients of settlement
- Shear strength
- Liquefaction potential.

Safe bearing capacity is the allowable pressure or load intensity that can be placed on the soil stratum at foundation level without causing any damage to the foundation or superstructure.

The downward movement or settlement of the foundation under loaded conditions takes place for various reasons like elastic compression of granular soil, consolidation of cohesive soil, changes in pore water pressure due to the lowering of the groundwater level, underground vibration caused by earthquakes or equipment-generated force, movement of soil below the foundation, and increased effective pressure induced by compressible fill over ground, etc. Settlement of the foundation is an unavoidable phenomenon that is time-dependent. The foundation engineer has to compute that the magnitude of the settlement of the foundation system at design load and that of the computed settlement are within the allowable limit or settlement criteria. Settlement criteria of different structures vary according to the functional requirements of the building/structure.

The value of safe bearing capacity can be determined by the following steps:

1. To compute the ultimate bearing capacity of the soil structure below foundation level using Terzaghi's equation for bearing capacity. Properties of soil shall be the average value of underlying layers up to a depth of 1.5 to 2 times the width of the foundation. Allowable or safe bearing pressure (p) will be equal to the ultimate value divided by the appropriate factor of safety (may be 3 to 5).
2. To determine the settlement of soil strata, apply pressure the same as the safe bearing pressure (p) at foundation level. Determine the pressure distribution (2 vertical: 1 horizontal) to soil layers below the foundation up to the depth at which the pressure increase from the foundation is less than 10% of the existing effective overburden pressure at that depth. The depth at which the pressure increase is less than 10% will provide the total thickness (H) of soil to be evaluated in the settlement computation. Calculate the total settlement (S).

For example, the permissible total settlement of a building is 25 mm. The bearing pressure (p) obtained from Terzaghi's equation is 250 kN/sqm, and the computed settlement (S) of the soil stratum below the foundation is 50 mm at the applied load (p), i.e., 250kN/sqm.

In that case, the design safe bearing pressure (p_{design}) shall be 125 kN/sqm in order to meet the settlement criteria, 25mm.

Like bearing capacity, shear strength is another important property of soil structure. Soil cannot resist any tensile force like concrete or steel, so uplift is prevented by the mass of backfill on top of the foundation and the frictional resistance. Pile bearing capacity in granular soil depends on the internal friction of the soil.

Liquefaction study of the site is a specialized subject and is not discussed here. It is done by the geotechnical engineer and furnished in the recommendation of the soil report.

2.3 GEOTECHNICAL INVESTIGATION TESTS AND PROPERTIES OF SOIL

A geotechnical report includes a description of the test, test location plan, field data and laboratory results. It is summarized with a recommendation report.

Some of the major types of tests and the properties of soil that we get from geotechnical investigations and laboratory tests are presented in Table 2.1.

TABLE 2.1
Geotechnical tests and engineering properties of soil

SL No	Soil test definition	Properties of soil
1	Bore hole test	Description of soil/rock stratum; standard penetration test (SPT value at various depth = N) for direct measure of consistency and relative density and safe bearing pressure; groundwater level
2	Cone penetration test	Relative density; shear strength
3	Open trial pits and plate load test	Undisturbed soil sample below ground level; bearing pressure, subgrade modulus
4	Open trial pits and block vibration test	Dynamic parameters (shear modulus; coefficients of elastic uniform compression Cu, elastic uniform shear $C\tau$, elastic non-uniform compression $C\phi$ and elastic non-uniform shear $C\xi$)
5	Field permeability test in selected bore holes	Coefficient of permeability of soil
6	Menard pressuremeter test	Shear strength
7	Vane shear test	Shear strength; shear modulus (static); shear stress/strain (μ)
8	Soil resistivity test	Electrical resistivity of soil
9	Field California bearing ratio test	In-situ penetration test; values are used for road and pavement design

TABLE 2.1 (*Continued*)
Geotechnical tests and engineering properties of soil

SL No	Soil test definition	Properties of soil
10	Seismic refraction tests i) Cross hole test ii) Down hole test	Dynamic parameters (P-wave velocity; modulus of elasticity E, Poisson's ratio; shear modulus coefficients of elastic uniform compression Cu, elastic uniform shear $C\tau$, elastic non-uniform compression $C\phi$ and elastic non-uniform shear $C\xi$; dynamic shear modulus G; damping)
11	Laboratory test of soils samples	
	a) Natural moisture content	To determine bearing pressure and settlement
	b) Bulk and dry density	As above
	c) Particle size analysis • Sieve analysis • Hydrometer test	Grain size distribution curve
	d) Liquid limit, plastic limits, plasticity index, shrinkage limit, liquidity index, relative consistency	Atterberg limits; index properties
	e) Unconfined compression test	Cohesion (c) and angle of friction (ϕ)
	f) Shear test by triaxial compression	Cohesion (c) and angle of friction (ϕ)
	g) Direct shear test	Shear strength
	h) Consolidation test	Coefficient of consolidation
	i) Specific gravity of soil/rock	To determine bearing pressure and settlement (soil)
	j) Swelling characteristics	Swelling pressure and free swell index
	k) Crushing strength of rock i) Soaked condition ii) Unsoaked condition	Safe bearing capacity and shear strength.
	l) Chemical analysis for i) Soil ii) Subsoil water	to determine pH value, sulfate, chloride, carbonate and organic content
	m) Proctor density test	To determine water content-dry density relationship to get maximum dry density of the soil at optimum moisture content

2.4 SELECTION OF TYPE OF FOUNDATION

The selection of the appropriate type of foundation depends on the magnitude of load on the column pedestal, soil bearing capacity, settlement limitations and other criteria, for example, availability of materials and construction equipment, time of construction, and cost-effective solutions.

The role of the foundation engineer is to provide a safe foundation system, which can bear the load satisfying strength design limits and distribute the applied load (from the column pedestal) in the form of uniformly distributed compression and shearing load on the soil stratum below the foundation. The soil structure can only take vertical compression and shear; it cannot take tension. For example, an isolated footing transforms the concentrated load applied on the column pedestal into a distributed load spread on the soil. Similarly, a combined footing or raft foundation converts the point loads into a distributed load spread onto the soil stratum below founding level.

Pile and well foundations are called deep-seated foundations because they transfer the load into the deep underground soil stratum. This applied load is resisted by shaft friction and tip bearing compression of piles.

All kinds of foundations can be broadly divided into two types – shallow foundations directly resting on soil and the other type is deep-seated foundation – pile or caissons (well) buried deep underground.

Building column foundations are normally designed with isolated spread footing or raft. The shape of the foundation depends on the column load and effective area required to be in contact with the soil. If the vertical load is predominant, the footing should be square-shaped in plan. For moment-carrying foundations, the shape may be rectangular in plan, having the longer side parallel to the direction of moment and base shear. The raft foundation is a plate resting on soil. The subgrade modulus of soil resists the overburden load similar to a set of springs holding the load on a cushion.

Strip footing or combined footings are used when the area required in the case of isolated footings overlaps. Combined footings and Raft foundations are preferred to control differential settlement between columns.

Basement-type substructures are generally used for large reservoirs and pump houses below ground, multi-story houses with an underground car parking floor, or cable vault structures, etc. In this type of substructure, the foundation raft is a mat or thick slab housing all the internal columns and peripheral walls. This basement structure should be designed as a water-retaining structure.

There are other types of below-ground structures like concrete tunnels for cables, conveyors, pipes and transportation systems.

Foundations for large-diameter tanks are designed with Ring wall-type foundations, Sand pad on soil or Mat foundations resting on piles.

Well or caisson foundations are used for intake pump houses and bridge piers on riverbeds.

Types of foundations are selected depending on the magnitude of load, i.e., load on the foundation pedestal and allowable soil bearing capacity satisfying the settlement criteria specified for the superstructure. In the case of a project site situated on soft or medium soft soil or loose sand layers at the upper level, the allowable bearing pressure may be lower than the required soil contact pressure, and the upper stratum fails to carry the foundation load. Pile foundations are chosen in such cases to transfer the load onto the deep stratum by shaft friction and tip bearing resistance of piles.

The types of piles commonly used are as follows:

- Bored cast-in-place
- Driven pile – steel pile or precast concrete pile
- Driven steel casing and then cast-in-concrete
- Driven – Pre-stressed precast pile

Selection of a pile foundation type for a structure should be based on the soil condition, foundation loading, and final performance criteria. While choosing the appropriate type of pile, the engineer should also consider the method of construction, availability of pile driving equipment, and facilities for manufacture, supply, and time of installation.

2.5 PROTECTION OF FOUNDATION AGAINST SUBSOIL CORROSION

The factors influencing the corrosion attack on foundations and piles are high sulfate and chloride concentrations, including pH of the soil, movement of the groundwater level and the density of concrete of the pile and foundation. The contact of sulfate ions in groundwater with Portland cement causes an expansive chemical reaction that may often lead to cracking and spalling of concrete. The concrete section loses its structural capacity. One method of reducing sulfate attack is to use dense concrete with sulfate-resisting cement or ordinary Portland cement mixed with pozzolanic material (ground granulated blast furnace slag).

Chloride ions in soil are also responsible for corrosion. Instead of affecting the concrete, chlorides cause corrosion of reinforcement steel, resulting in expansion and bursting of concrete as products of steel corrosion (carbonation) form around reinforcement bars.

The corrosion of steel reinforcement induced by chloride ions is one of the main causes of the degradation of concrete structures.

The permissible limits of solids in water should be as given below:

- Sulfates (as SO_3) – 400 mg/liter as per IS 456 [For exposure class in US standard, refer to Table 19.3.1.1 and 19.3.2.1 ACI 318–19]
- Chloride (as Cl) – 500 mg/liter (< 500ppm)

For ordinary soil, where the sulfate and chloride content are within permissible limits, protection of concrete is provided by adequate cover thickness and a blinding concrete layer on the ground for slabs, grade beams, walls and foundations.

Where the limits of sulfates and chloride exceed permissible limits, the use of the following materials, as per code and specification, may control corrosion:

a) Sulfate-resistant cement (Type V, ASTM C150 or equivalent).
b) Corrosion-resistant steel (CRS) bars or epoxy-coated or zinc- and epoxy-coated reinforcement bars.
c) Protection paint – bitumastic or bitumen emulsion or epoxy paint on surfaces in contact with soil and on blinding concrete.
d) Tanking membrane – polythene sheet bonded to bitumen or polymer adhesive or waterproofing geotextile liner
e) Laying polythene sheet (250 micron) over blinding concrete for ground floors and slabs on grade.

REFERENCES

American Concrete Institute (ACI). *Building Code Requirements for Structural Concrete (ACI 318–19)*. USA: Farmington Hills, MI 48331, 19th ed., 2019.

Bureau of Indian Standards (BIS). *Indian Standard Plain and Reinforced Concrete – IS 456:2000*. New Delhi: 9 Bahadur Shah Zafar Marg, Pin 110002, 4th Revision, 2000.

3 Structural concrete

3.1 GRADE OF CONCRETE AND APPLICATION

The grade of concrete is defined by its characteristic strength.

In many countries, the grades of concrete are designated by prefixing “M”; for example, M20, M25, etc. In this designation, M refers to the concrete mix, and the number is the specified characteristic compressive strength of a 15 cm cube at 28 days in N/mm^2. The grade of concrete is also defined by prefixing “C”. The ‘“C”’ stands for “compressive strength – cylinder strength”.

For example, **M25** stands for cube’s characteristic strength and **C25** indicates cylindrical strength after a curing period of 28 days, which represents 25 N/mm^2.

- M30 = 30 N/mm^2 (cube strength)
- C30 = 30 N/mm^2 (cylinder strength)

In American standards, the strength of concrete is designated by cylinder strength. The strength test shall be the average of the strengths of at least two 150 by 300 mm cylinders or at least three 100 by 200 mm cylinders made from the same sample of concrete and tested at 28 days of age or at the test age designated for determination of **f′c**, i.e., specified compressive strength of concrete.

Minimum cylinder compressive strength = 0.8 times specified for 15 cm cubes.

Requirements for concrete mixtures are based on the philosophy that the concrete structure should provide:

- Durability requirements based on exposure classes
- Structural strength requirements

The codes define the minimum grade of concrete to be used under different exposure conditions. The following tables are prepared as a guideline for the selection of grade in general applications.

Exposure class:

1. Mild – surface protected against weather or aggressive conditions, except coastal areas
2. Moderate
   - sheltered from freezing
   - exposed to rain and condensation
   - continuously underwater
   - in contact with aggressive soil or groundwater
   - sheltered from saturated salt air in coastal areas

DOI: 10.1201/9781003618119-3

TABLE 3.1
Minimum grade of concrete (cube strength) by exposure class

SL No	Application	Exposure class	Concrete grade designation	Compressive strength at 28 days (cube strength) N/mm²
1	Blinding concrete or levelling concrete on soil below foundation	Mild	M20	20
		Moderate	M25	25
		Severe	M30	30
		Very severe	M35	35
		Extreme	M40	40
2	Foundation and substructure	Mild	M25	25
		Moderate	M25	25
		Severe	M30	30
		Very severe	M35	35
		Extreme	M40	40
3	Superstructure – framing members, beams, columns, slabs, walls, stairs and all	Mild	M20	20
		Moderate	M25	25
		Severe	M30	30
		Very severe	M35	35
		Extreme	M40	40
4	Cast-in piles and caisson	Mild	M25	25
		Moderate	M25	25
		Severe	M30	30
		Very severe	M35	35
		Extreme	M40	40
5	Precast – non-pre-stressed driven piles, drilled shafts	All	M30	30
6	Precast – pre-stressed driven piles	All	M40	40
7	Water-retaining structure	Mild	M25	25
		Moderate	M25	25
		Severe	M30	30
		Very severe	M35	35
		Extreme	M40	40
8	Special structural walls – blast walls, etc.	All	M40	40

3. Severe
   - exposed to freezing and thawing
   - exposed to severe rain and condensation
   - completely immersed in seawater
   - In contact with aggressive soil or groundwater
   - exposed to coastal area

4. Very severe
   - exposed to severe freezing
   - exposed to corrosive fumes
   - exposed to seawater spray
   - In contact with aggressive soil or groundwater
   - exposed to coastal area
5. Extreme
   - surface exposed to tidal zone
   - in contact with liquid or solid aggressive chemicals

TABLE 3.2
Minimum grade of concrete (cylinder strength) by exposure class – ACI 318–19 (Chapter 19)

SL No	Application	Exposure class	Minimum specified compressive strength of concrete f′c psi
1	Blinding concrete or levelling concrete on soil below foundation	FO, S0, W0,C0	2500
		F1, S0, W1,C1	3500
		F2, S1,W2,C1	4000
		F3,S2,W2,C2	4500
		F3, S3,W2,C2	5000
2	Foundation and substructure	FO, S0, W0,C0	3000
		F1, S0, W1,C1	3500
		F2, S1,W2,C2	4000
		F3,S2,W2,C2	4500
		F3, S3,W2,C2	5000
3	Superstructure – framing members, beams, columns, slabs, walls, stairs and all	FO, S0, W0,C0	2500
		F1, S0, W1,C1	3000
		F2, S1,W2,C2	4000
		F3,S2,W2,C2	4500
		F3, S3,W2,C2	5000
4	Cast-in piles and caisson	FO, S0, W0,C0	3000
		F1, S0, W1,C1	3500
		F2, S1,W2,C2	4000
		F3,S2,W2,C2	4500
		F3, S3,W2,C2	5000
5	Precast – non-pre-stressed driven piles, drilled shafts		4000
6	Precast – pre-stressed driven piles		5000
7	Water-retaining structure	FO, S0, W0,C0	3000
		F1, S0, W1,C1	3500
		F2, S1,W2,C2	4000
		F3,S2,W2,C2	4500
		F3, S3,W2,C2	5000
8	Special structural walls – blast walls, etc.		5000

Exposure class:

1 F – exposed to freezing and thawing
F0 – Not exposed to freezing and thawing
F1 – Exposed to freezing and thawing with limited exposure to water
F2 – Exposed to freezing and thawing with frequent exposure to water
F3 – Exposed to freezing and thawing with frequent exposure to water and deicing chemicals
2 S – In contact with soluble sulfate ion in soil or water
S0 – Dissolved sulfate ($S0_4^{2-}$) in water, ppm < 150
S1 – 150 <= ($S0_4^{2-}$) < 1500 or seawater
S2 – 1500<= ($S0_4^{2-}$) <= 10,000
S3 – ($S0_4^{2-}$) > 10,000
3 W – In contact with water
W0 – Concrete dry in surface
W1 – Concrete in contact with water where low permeability is not required
W2 – Concrete in contact with water where low permeability is required
4 C – Pre-stressed or non-pre-stressed concrete that requires additional protection of reinforcement against corrosion
C0 – Concrete dry or protected from moisture
C1 – Exposed to moisture but not to an external source of chlorides
C2 – Exposed to moisture and an external source of chlorides from deicing chemicals, salt, brackish water, seawater or spray from these sources.

3.2 MATERIALS AND ADMIXTURES

The materials used for preparing cement concrete mix are as given below:

- Cement
- Water
- Aggregate
- Admixture
- Reinforcement steel
- Formwork

All materials used for design and construction should conform to national standard codes and standards prevailing in the country, unless there is a more stringent requirement specified in contract specifications. A brief description of the above materials is presented below for the designer's reference.

3.2.1 Cement

Cement is the binding material in the cement concrete mix and mortar for construction. The water-cement paste also fills up the pores between fine and coarse aggregates to make a dense concrete product. In a dense concrete mass, the ingress of corrosive air-water is less; hence, it is more durable.

The following types of cements are used for construction:

Indian Standard

a) Ordinary Portland Cement (OPC): IS 269
b) Portland pozzolana cement: IS 1489
c) Portland slag cement: IS 455
d) Hydrophobic cement: IS 8043
e) Rapid hardening Portland cement: IS 8041
f) High-strength ordinary Portland cement: IS 8112
g) Super-sulfated cement: IS 6909

American Standard

a) Portland cement: ASTM C150
b) Blended hydraulic cements: ASTM C595
c) Expensive hydraulic cement: ASTM C 845
d) Hydraulic cement: ASTM C 1157
e) Fly ash and natural pozzolan: ASTM C 618
f) Ground-granulated blast-furnace slag: ASTM C989
g) Silica fume: ASTM C 1240

3.2.2 Water

Water is the primary material used for preparing cement concrete and masonry work construction. It is used for washing and cleaning coarse aggregates, preparing water-cement paste for concrete mix and mortars, soaking of bricks and masonry units and curing. Water shall be clean, soft and potable – free from organic materials, oil, acid, salt and visible floating matters which are harmful for concrete mix.

Limitation of impurities in construction water is given below:

a) Chloride ions	<500 ppm
b) Sulfate SO_3	<500 ppm
c) Alkali carbonates and bicarbonates	<750 ppm
d) Other dissolved salts	<2000 ppm

Total dissolved salts a + b + c + d < 3000 ppm

The pH value of water shall generally be not less than 6.

3.2.3 Aggregate

Aggregates should comply with the requirements of applicable Codes of Practice (IS: 383 / ASTM C33 for normal weight and ASTM C330 for lightweight). Aggregates should be hard and dense, natural or crushed gravel or crushed rock – quartzite, sandstone, gravel, granite, basalt and limestone.

Aggregate sizes between 4.75 mm and 150 mm are termed as coarse aggregate. Aggregates smaller than 4.75 mm (within the grading limits set in codes of practice) are termed as fine aggregate. Aggregates shall be free from earth, shale, decomposed matters and other impurities likely to affect the durability of concrete. Aggregates shall not contain any materials that may react with alkalis in the aggregate itself or in the cement, or in mixing water in contact with finished concrete and mortar.

The acid-soluble chloride content as chloride ion in aggregate, expressed as a percent by mass (BS 812 Part 117), should not exceed the following limits:

Coarse aggregate	0.03%
Fine aggregate	0.06%

Water absorption of aggregate should not exceed 3%.

Recommended sizes of coarse aggregate are shown in Table 3.3.

Fine aggregates are natural sand. Natural sands are available in fine and coarse grain from various sources. In such cases, these two types may be combined to meet the grading requirement. Due to the scarcity of natural sand or river sand in some places, manufactured sand (M sand), which is prepared in combination with crushed stone and natural sand, may also be used.

3.2.4 Admixture

The admixture shall conform to relevant codes of practice (IS: 9103 / ASTM C494). It improves the properties of concrete, including setting time and design strength. The quantity of cement can be lowered in a design mix using admixture without sacrificing characteristic strength. Hence, the possibilities of drying shrinkage and cracks on the surface of concrete are also reduced. Admixtures containing chlorides should not be permitted.

The following types of admixtures are generally used:

a) Air-entraining admixture (ASTM C 260)
b) Water-reducing admixture (ASTM C 494, Type A)
c) Water-reducing and -retarding admixture (ASTM C 494, Type D)
d) Water-reducing high-range admixture, super-plasticizer (ASTM C 494, Type F or G, ASTM C 1017, Type 1 or 2)
e) Accelerating admixture (ASTM C 494, Type C)

TABLE 3.3
Recommended aggregate sizing for concrete items

Item	Max. size (mm)
Concreting very narrow space	12
Reinforced concrete except foundation	20
Foundations and plain cement concrete	40
Mass concrete	80
Mass concrete in very large structure	150

Special admixtures as corrosion inhibitors are also used, where necessary.

The use of ground granulated blast furnace slag (GGBS) with Portland cement enhances durability, provides protection against sulfate attack and also protects steel reinforcement against chloride ingress. GGBS is blended with cement in the mixture during concrete production. GGBS should conform to the requirements of BS 6699 or ASTM C 989 (grade 120).

3.2.5 Reinforcing steel

The following types of reinforcement bars are used in reinforced concrete structures:

Indian Standards

a) Mild steel bars (IS: 432) – yield stress 240 N/mm^2
b) High yield strength deformed bars (IS: 1786) – yield stress 415 N/mm^2 and 500 N/mm^2.
c) CRS bars (CRSI) – ASTM; yield stress above 500 N/mm^2

American Standards

a) ACI – 318
b) Carbon Steel: ASTM A615 M
c) Low-alloy steel: ASTM A 706M

Bars in contact at crossing points are securely tied together at all points with 20-gauge annealed soft iron wires. Fabrication of bars shall be done as per guidance furnished in IS: 2502 or ACI 315. All bars should be cold bent unless required otherwise. Welded splices should be avoided as far as possible.

3.2.6 Formwork

Forms for footings, trenches, pits, walls, columns, etc. which are below ground are made of mild steel plate board formed by thin sheets and welded ribs, wooden planks made to shape and plywood boards with wooden frames. All these boards are made of standard size so that these can be used as modular units for different shapes of concrete structures. Bituminous paints are applied on the contact face of the ply board to make it useful for many times. Plastic sheets are also used to keep the surface free from cement slurry.

In general, superstructure formwork is made of steel or coated (paint or plastic) plywood panels. The contact face is covered with plastic sheets. These panel boards are framed with nailed wood runners, which carry the weight of green concrete. Steel tube sections joined with removable clamps and bolts are used to support formwork for slabs and beams at floors. The diameter and spacing of tubular structured supporting props are designed to carry the weight of fresh concrete and construction load (shall not be less than 300 kg/sqm). Timber props (sal wood) and bamboos are also used as prop members for small structures.

For special structures like deep underground excavation and waterfront structures, steel sheet piles are used. These sheet piles are made to form diaphragm walls braced with steel members – joists or channel sections. All such formworks are designed to retain earth pressure and construction loads.

Slabs over large halls at high altitude are cast on steel metal deck forms. These are made with cold-formed steel (0.8 to 1.2 mm thick) arranged to shape like corrugated roof sheets but with deep valleys (up to 175mm). These valleys are spaced 140 to 200 mm apart. Sheets are laid across the roof/floor beams so that valleys span over supporting beams. The sheets are stitched together side by side and fixed to the supporting beam by metal fasteners. The use of such deck sheets is popular in the construction of bridges and industrial buildings.

For the construction of chimney, silos, elevator shafts in high-rise buildings and conveyor-supporting towers, the slip form method of construction is used. In this method, a collapsible type steel shuttering system is used. The shuttering unit is assembled on the ground, bracing the first lift of the concrete shell or structure. This slip form shuttering unit is mechanically jacked slowly upward at a controlled rate until the required elevation is reached. There is a working deck in and around the shuttering units, which also moves up. The starter section of the shell, which is cast on the foundation base, has reinforcement bars on both sides projecting up and a set of jack rods placed at intervals in between the re-bars along the center of the wall. This entire unit is clamped on the jack rod like a floating deck. These jack rods are made of high-strength steel and allow the slip form structure to climb upwards, driven by an oily pump machine installed on the ground. The power cable and supply pipes go up with the deck. The workmen do reinforcement placing, binding and pouring of concrete mix supplied by the concrete pump hose ahead of the slip form shutter moving up. The supply of all materials and working personnel is provided by cranes and winch-operated lifts erected inside at the center of the shell. The concrete mix is designed with proper admixtures so that the slip form shutter slides smoothly up, leaving the hardened concrete below. The hardened shell structure serves as a supporting tower until the total height is reached. The rate of concreting and jacking of slip form is controlled as per the calculated strength of cast-in concrete. This work should be done by specialist agencies.

3.3 METHOD OF CONSTRUCTION DOCUMENT

The method of construction is a document that defines details of work steps and procedures to be followed at the construction site. The construction engineer should prepare this document and submit it for the owner's approval.

The method of construction document is prepared in standard engineering procedure formats of the construction company. It covers the following steps in general:

a) Cover sheet – Title block showing the work to be done, for example: "Placement of Concrete and Curing".
b) Scope of work
c) Abbreviation, if any
d) References – Drawings, technical specifications and quality control procedures
e) Responsibilities (site engineer and construction manager's task)

f) Resources

Materials: Ready mix concrete (grade, slump, etc.), potable water for curing, hessians and polythene sheets

Plant and equipment: Batching plant, transit mixer trucks, concrete pumps, mobile crane, concrete vibrator and finishing tools and laboratory equipment for testing and quality control work.

Manpower: Trained employees for field and laboratory work

g) Methodology: In this part, work procedures from supply to placement and up to curing are defined.

h) Quality control managements

i) Safety requirements

REFERENCES

American Concrete Institute (ACI). *Building Code Requirements for Structural Concrete (ACI 318–19)*. USA: Farmington Hills, MI 48331, 19th ed., 2019.

Bureau of Indian Standards (BIS). *Indian Standard Plain and Reinforced Concrete – IS 456:2000*. New Delhi: 9 Bahadur Shah Zafar Marg, Pin 110002, 4th Revision, 2000.

4 Codes and standards

4.1 NOTES ON DESIGN AND CONSTRUCTION

The material specifications, rules, standards and reference codes are provided in the Standard Notes. These notes are considered a part of the design drawing. The standard note drawings are accompanied by typical construction details, which are provided in Chapter 6.

Lists of Standard Notes given below are commonly used for construction.

A. General

1. All dimensions are in millimeter except floor elevations and plant grids, which are in meter unless stated otherwise.
2. All elevations are with respect to the main plan building ground floor finish level as EL.(+/−) 0.00 M, which corresponds to RL____ (reduced level with respect to O.S. datum/mean sea level)
3. All drawings should be read in conjunction with the terms, conditions and specifications of the contract.

B. Foundation medium

4. The net allowable bearing pressure considered in the design may be noted as follows:
   a) Foundation resting on hard rock – _______ kN/sqm.
   b) Foundation resting on virgin soil – _______ kN/sqm.
   c) Foundation resting on fill material – _______ kN/sqm.
5 Foundations resting on piles:
   Pile type/pile diameter/length – ___ / ___ mm/___ M
   Design capacity: axial compression/tension/lateral
6. The following should be noted for different types of foundation medium:
   a) For foundations resting on rock – No foundation should be placed on a sloped rock surface. Proper benching should be done; where rock is not available at the bottom of the foundation, the space between rock level and the bottom of the foundation should be filled up by structural mass concrete of grade C20 (or M25). The minimum thickness of mass concrete filling should be 200mm.
   b) For foundations resting on virgin soil – The minimum depth of foundation below virgin soil should be 1.0 meter. Where virgin soil is not available at the bottom of the foundation, the space between virgin soil of required depth and the bottom of the foundation should be filled up by engineering fill (grade crushed stone) material compacted to a minimum of 95% of the maximum dry density (modified

DOI: 10.1201/9781003618119-4

proctor) so as to achieve a minimum allowable net bearing capacity of 200kN/sqm.

c) Foundations resting on fill material – A minimum of 0.5 M below the foundation should be of compacted subgrade to a minimum of 95% of the maximum dry density (modified proctor).

d) All excavated formations should be checked by an experienced geotechnical engineer to ensure uniformity and suitability for the founding medium. Any suspected material, whether made ground or weak virgin soil, should be removed and replaced by engineering fill material.

C. Concrete work

7. The concrete work should be in accordance with American Standard (ACI 318)/British Standard (BS 8110 Parts 1 & 2)/Eurocode/Indian Standard (IS 456).
8. All cement should be ordinary Portland cement/OPC with GGBS (ground granulated blast furnace slag)/sulfate-resistant cement. [*The designer should mention the actual type of cement to be used].*
9. All non-structural concrete (plain cement concrete) used for levelling and blinding layer will be of grade ___ (C10/M15).
10. All structural plain concrete designated as structural mass concrete used for should be of grade ___ (C15/M20).
11. All structural reinforced concrete for foundations, pits, trenches, ground floor slabs and other substructures should be of grade . . . (C/M)
12. All structural reinforced concrete for superstructure work above ground floor should be of grade ___ (C/M)
13. All structural reinforced concrete should have a minimum cement content and maximum free water/cement ratio as follows:

    Grade of concrete/minimum cement content/maximum free water cement ratio *(designer to mention).*
14. Aggregate for all concrete work should be graded conforming to (BS/ACI/IS code) with 20 mm as the maximum size of coarse aggregate.
15. All reinforcement bars for concrete work should be of the following types conforming to BS/ACI/IS code ___:
    a) High yield strength hot rolled deformed bars of grade . . . (minimum yield strength ___)
    b) Mild steel bars of grade ___ (minimum yield strength ___)
16. All embedded mild steel plates and sections should of be grade (ASTM A36/S275/IS2062 fy 250MPa) conforming to ASTM/BS/IS code
17. Clear cover to reinforcement should be as follows:

18 Provide 20 mm chamfer at all exposed edges of concrete unless noted otherwise.

19 Construction joints should be located in design drawings.

20 All reinforcement steel bar splices and anchorage/embedded length should be provided considering all bars in tension, unless mentioned otherwise.

SL no	Item	Bottom	Sides	Top	Ends
	Substructure work				
i	Foundation	75	50	50	50
ii	Columns, pedestals	50	50	50	50
	Grade beams, tie beams	50	40	40	50
iii	Trenches, pits, walls, etc. in contact with soil/groundwater	50	50	50	50
iv	Cable duct bank	75	75	75	75
v	Equipment foundations	75	50	50	50
vi	Ground floor slab/slab on grade	50	50	20	50
	Superstructure work				
i	Columns		40		
ii	Beams	35	35	35	50
iii	Slabs/walls	20	20	20	40
iv	Lintel, chajja, etc.	20	20	20	25
v	Precast concrete	20	20	20	20

Note: This table shows normal cover followed for structures in non-corrosive environments. The covers for foundations and structures in contact with aggressive soils (high sulfate and chloride contamination) and exposed to salt air in coastal areas should not be less than 75 mm.

21 Splicing of bars should be kept to a minimum as possible using the maximum available length.
22 The splices should be staggered, and in no case should more than 50% of bars be spliced at one section.
23 Minor adjustments of reinforcements may be done at the site to clear pockets, bolts, openings, etc.
24 No backfill should be placed against concrete structures until the concrete has been in place for 14 days or has attained 80% of specified compressive strength.
25 Grout under column base plates should be cement-based non-shrink grout of compressive strength higher than the grade of the supporting pedestal.
26 Reinforcement bars should be equally spaced where no specific spacing is given.
27 Concrete finish floor surfaces should have a steel troweled finish unless noted otherwise.
28 All concrete surfaces other than floor surfaces should have a rough float finish.
29 Concrete sampling, testing, and compliance should be in accordance with relevant American/British/Indian standards or other international codes

4.2 REFERENCE LIST OF CODES AND STANDARDS

Following codes and standards are used for design and construction for civil and reinforced cement concrete structural work.

American Standards

- ASCE 7 Minimum Design Loads for Buildings and Other Structures
- Uniform Building Code 1997 (UBC 1997)
- Uniform Plumbing Code (UPC)
- Construction Industry Research and Information Association (CIRIA) C577, guide to construction of reinforced concrete in the Arabian Peninsula.
- ACI 318, American Concrete Institute, Building Code Requirements for Structural Concrete and Commentary
- ACI 543R, Design, Manufacture and Installation of Concrete Piles
- ACI 350, Code of Practice for Environmental Engineering Concrete Structures
- CRSI, Concrete Reinforcing Steel Institute, Manual of Standard Practice
- AISC, American Institute of Steel Construction, Manual of Steel Construction
- ACI 530.1, American Concrete Institute, Specification for Masonry Structures
- AWS D1.1, American Welding Society Structural Welding Code for Steel
- ASTM, American Society for Testing and Materials Standards (as applicable)
- NFPA 850, National Fire Protection Association, Recommended Practice of Fire Protection for Electric Generating Plants
- AWWA, American Water Works Association
- AASHTO, American Association of State Highway and Transportation Officials.

British Standards

- Euro Code 8 for Design Provisions for Earthquake Resistance Structures
- British Standard, BS 6399 Part 1, Part 2 & Part 3 for Building Loads and Wind Forces
- British Standard, BS 8110 Part 1 & Part 2 for Structural Use of Concrete
- British Standard, BS 449–2 Specification for the Use of Structural Steel in Building
- British Standard, BS 4449 for Concrete Reinforcing Steel Bars
- BS 8007 Design of Concrete Structures for Retaining Aqueous Liquids

Indian Standards

- IS: 456 Plain and Reinforced Concrete – Code of Practice
- SP 16: Design Aid for Reinforced Concrete to IS: 456
- IS: 875 Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures
    - Part 1 Dead Loads – Unit weights of building materials and stored materials.
    - Part 2 Imposed Loads
    - Part 3 Wind Loads
    - Part 4 Snow Loads
    - Part 5 Special Loads and Combinations

- IS: 1893 Criteria for Earthquake-Resistant Design of Structures
  - Part 1General Provisions and Buildings
  - Part 2 Liquid-retaining tanks – Elevated and ground supported
  - Part 3 Bridges and retaining walls
  - Part 4 Industrial structures including stack like structures
  - Part 5 Dams and embankments
- IS: 2911 Code of Practice for Design and Construction of Pile foundation.
  - Part 1/Section 1 Driven cast-in-situ concrete piles
  - Part 1/Section 2 Bored cast-in-situ piles
  - Part 1 / Section 3 Driven precast concrete piles
  - Part 1 / Section 1 Bored precast concrete piles
- IS: 3370 Concrete Structures for storage of liquids – Code of Practice
  - Part 1 General requirements
  - Part 2 Reinforced concrete structures
  - Part 3 Pre-stressed concrete structures
  - Part 4 Design table
- IS: 2502 Code of Practice for Bending and Fixing of Bars for Concrete Reinforcement
- SP:34 Handbook of Concrete Reinforcement and Detailing
- IS: 800 General Construction in Steel – Code of Practice
- IS:2974 C(all parts).ode of Practice for Design and Construction of Machine Foundations
- IS:1343 Pre-stressed Concrete – Code of Practice
- IS:4998 Criteria for Design of Reinforced Concrete Chimneys
- Relevant Codes & Standards followed by Indian Road Congress (IRC)
- IRC: 5 Standard Specifications and Code of Practice for Road Bridges
  - Section I: General features of design
- IRC: 6 Standard Specifications and Code of Practice for Road Bridges
  - Section II: Loads and Stresses
- IS:6403 Code of Practice for Determination of Allowable Bearing Pressure and Shallow Foundation
- IS:8009 (Part-I) Code of Practice for Calculation of Settlement of Foundation subject to Symmetrical Vertical Loads for Shallow Foundation
- IS:2212 Code of Practice for Brickwork
- IS:2250 Code of Practice for Preparation and Use of Masonry Mortar
- IS:1661 Code of Practice for Cement and Cement-lime Plaster Finish on Walls and Ceilings
- IS:12118 Specification for Two Parts Polysulfide-based (Part-I & II) Sealants
- IS:2470 Code of Practice for Designs and Construction of Septic Tank for Small and Large Installations
- IS:3889 Centrifugally Cast (spun) Iron Spigot and Socket Soil Waste and Ventilating Pipes, Fittings and Accessories
- IS:1729 Sand Cast Iron Spigot & Socket Soil, Waste and Ventilating Pipes and Accessories.
- IS:2527 Code of Practice for Fixing Rain Water Gutters and Down-pipes for Roof Drainage
- IS: 3414 Code of Practice for Design and Installation of Joins in Buildings.

5 Workout examples by ACI and IS Code method

This chapter will cover design examples of primary members used in an RCC building superstructure and foundation. All the members are designed in accordance with American Standard code (Load factor method) and Indian code (Limit State method).

5.1 DESIGN OF PILES – BORED AND DRIVEN

5.1.1 Driven pile in clay

(Reference: Workshop manual Vol I – NHI US Department of Transportation; Foundation analysis and design by J E Bowels; Pile design and construction practice – Tomlinson.)

Pile type: Precast pre-stressed concrete pile
Pile diameter, d = 600 mm.
Length of pile, Lp = 28 M.
Pile cut-off level = –2.5 M.
Finish-grade level, FGL = 0.0 M
Groundwater level, GWL = 0.0 M (assumed for design)

DOI: 10.1201/9781003618119-5

5.1.1.1 Sketch

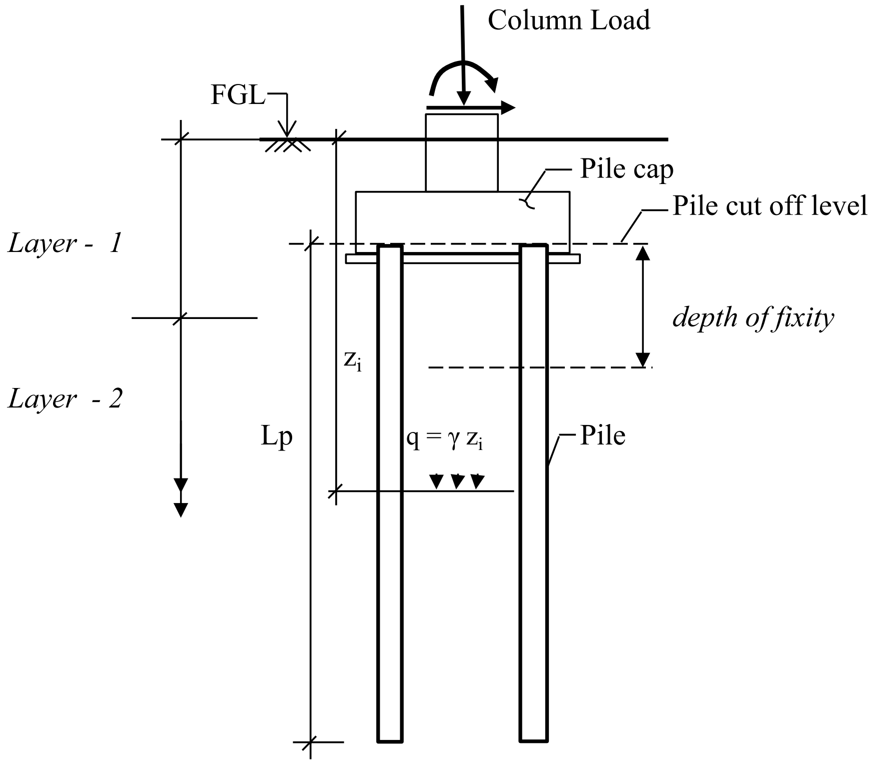

FIGURE 5.1 Pile in soil

5.1.1.2 Soil parameters

TABLE 5.1
Description of soil from bore hole data

Layer mark	Description	Thickness (D) in meter	SPT value (N60)	Remarks
1	Gray soft silty clay	5	5	
2	Gray medium-stiff sandy silt	6	16	
3	Gray medium-dense silty sand with clay	8	22	
4	Gray medium-dense silt and fine sand with clay	4	25	
5	Gray stiff hard clayey silt with fine sand	17	32	Founding layer

TABLE 5.2
Soil properties from laboratory tests

Layers	γs kN/m³	c_u kPa	ϕ deg
Layer 1	15.16	48.22	6.6
Layer 2	18.5	51.65	7.32
Layer 3	18.5	21.66	29.48
Layer 4	20	57.21	8.77
Layer 5	20	96.19	8.05

A reference bore hole data are given below representing description of soil stratum

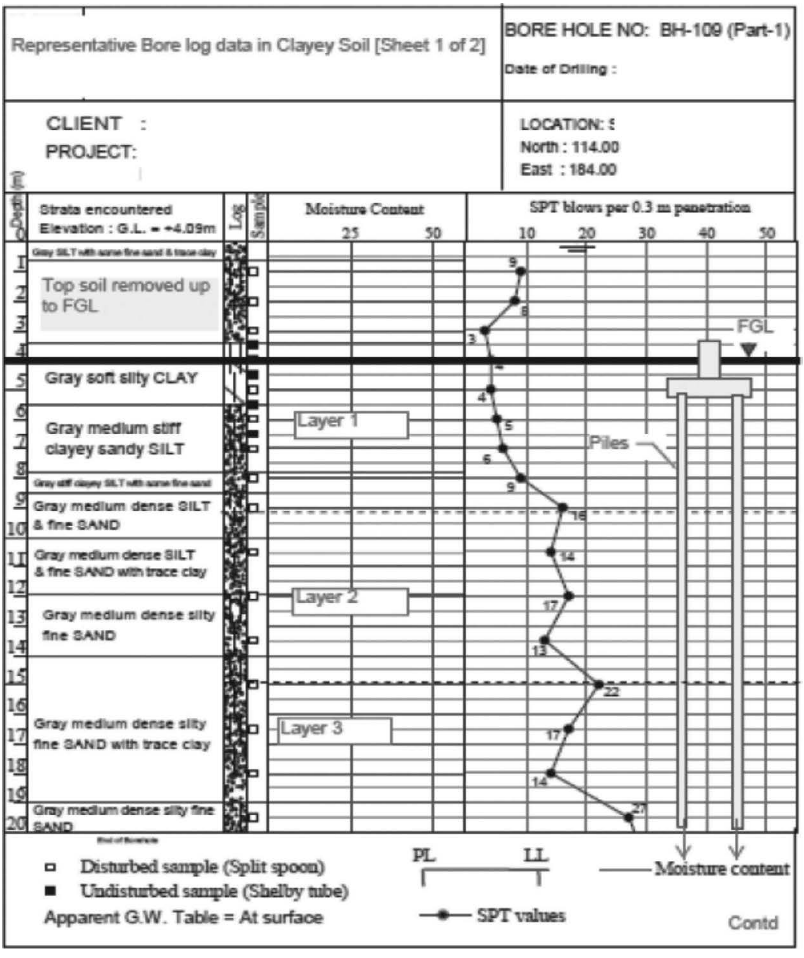

FIGURE 5.2 Bore hole data – Sheet 1 of 2

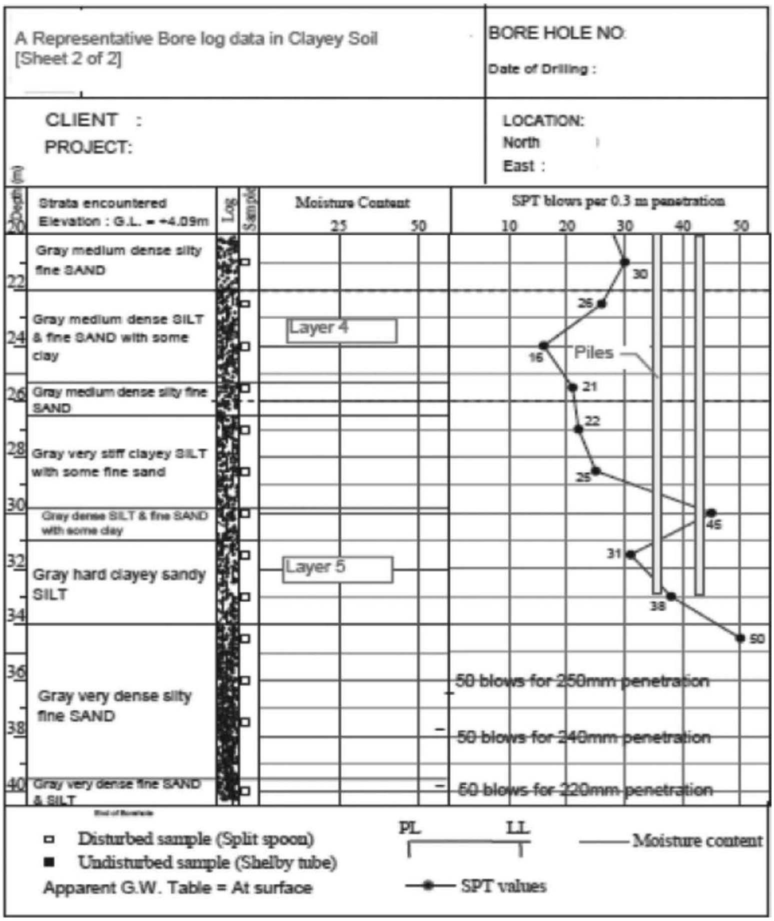

FIGURE 5.3 Bore hole data – Sheet 2 of 2

5.1.1.3 Pile capacity determination

a) *Nordlund method for pile bearing capacity in cohesionless soil (Workshop manual –9.7.1.1b)*

$Qu = (K.C_F.\ p_d.\ \sin\phi\ C_d\ D) + (\alpha_t.N'_q.\ A_t.p_t)$ (Eqn 5.1)

(Shaft resistance) + (Toe resistance)

where Qu = ultimate capacity of the pile in cohesionless soil

d = depth

D = embedded pile length

K = coefficient of lateral earth pressure at depth d

[refer to Table 9.2 (a) & (b) – Workshop manual]

C_F = 1 correction factor for K (1 for piles of uniform cross section]

[Figure 9.15 – Workshop manual]

p_d = effective overburden pressure at the center of depth increment, d

ϕ = soil friction angle

C_d = pile perimeter at depth d

α_t = dimensional factor (dependent on pile depth-width relationship) [Figure 9.16 – Workshop manual]
N'_q = bearing capacity factor
At = pile toe area
p_t = effective overburden pressure at the pile toe

b) *Total stress – α method for cohesive soil:*
[Workshop manual – 9.7.1.2a]
$Qu = (\alpha\, c_u\, As) + (Nc\, c_u\, At)$ (Eqn. 5.2)
(Shaft resistance) + (Toe resistance)
where Qu = ultimate capacity of the pile in cohesive soil
α = adhesion factor [refer to Figure 16.14 – Bowel]
c_u = undrained shear strength of soil (1/2 q_u; q_u = undrained compressive strength)
Nc = 9 bearing capacity factor
As = pile-soil surface area from the pile perimeter and length
At = area of the pile toe

[Designer should choose the applicable formula of shaft resistance and toe resistance according to the property of soil].

5.1.1.4 Axial capacity

TABLE 5.3
Pile bearing capacity calculation (axial capacity)

Layer	Calculation	Pile capacity (kN)	Reference
1	*Gray soft silty clay – cohesive soil*		
	N60 = 5 ϕ = 6.6 deg		
	Embedded length D = 2.5 m (length below pile cut off level)		
	$\gamma_{sat} - \gamma_w = 5.16\ kN/m^3$		
	Pile diameter, d = 0.6 m		
	c_u = 48.22 KPa		
	α = 0.91 (see Figure 16.14 – Bowels)		
	$As = 3.14 \times 0.6 \times 2.5 = 4.71\ m^2$		
	Shaft resistance (from Eqn. 5.2)		
	$Qu = (\alpha\, c_u\, As) = 0.91 \times 48.22 \times 4.71 = 207$ kN	207	
2	*Gray medium stiff sandy silt – cohesive soil*		
	N60 = 16 ϕ = 7.32 deg		
	Embedded length D = 6 m		
	$\gamma_{sat} - \gamma_w = 8.5\ kN/m^3$		
	Pile diameter, d = 0.6 m		
	c_u = 51.65 KPa		
	D/d = 10; α = 0.4 (refer to Figure 9.19 Workshop Manual)		
	$As = 3.14 \times 0.6 \times 6 = 11.3\ m^2$		
	Shaft resistance:		
	$Qu = (\alpha\, c_u\, As) = 0.4 \times 51.65 \times 11.3 = 233$ kN	233	

(*Continued*)

TABLE 5.3 (*Continued*)
Pile bearing capacity calculation (axial capacity)

Layer	Calculation	Pile capacity (kN)	Reference
3	*Gray medium-dense silty sand with clay – cohesionless soil* N60 = 22 ϕ = 29.48 deg Embedded, length D = 8 m $\gamma_{sat} - \gamma_w$ = 8.5 kN/m^3 Pile diameter, d = 0.6 m c_u = 21.66 KPa D/d = 13.33; α = 0.5 (refer to Figure 9.19 Workshop Manual) As = 3.14 × 0.6 × 8 = 15.07 m^2 V = volume of soil replaced by pile per m (m^3/m) = (3.14 × 0.6^2) / 4 = 0.283 m^3/m K = 1.15 for V = 0.093 and K = 1.45 for V = 0.930 [Table 9.2 a – Workshop Manual] So by interpolation, we get K = 1.22 for V = 0.283 m^3/m. C_F = 1 [Figure 9.15 – Workshop Manual] Critical depth for skin friction = 20 × pile diameter. *(The unit skin friction remains at a constant value below a penetration depth of 10 to 20 diameter of pile. Refer to "Pile design and construction Practice by Tomlinson".)* $d_{effective}$ = 20 × 0.6 – 2.5 – 6 = 3.5 m. p_d = 5 × 5.16 + 6 × 8.5 + 0.5 × 3.5 × 8.5 = 92 kN/m^2 sin ϕ = 0.492 C_d = 3.14 × 6 = 1.884 m *Shaft resistance:* Qu = soil friction = K C_F p_d sin ϕ C_d D = 1.218 × 1 × 92 × 0.492 × 1.884 × 8 = 831 kN.	831	
4	*Gray medium-dense Silty and fine sand with clay – cohesive soil* N60 = 25 ϕ = 8.77 deg Embedded length D = 4 m $\gamma_{sat} - \gamma_w$ = 10 kN/m^3 Pile diameter, d = 0.6 m c_u = 57.21 KPa α = 0.87 (see Figure 16.14 – Bowel) As = 3.14 × 0.6 × 4 = 7.536 m^2 *Shaft resistance:* Qu = (α c_u As) = 0.87 × 57.21 × 7.536 = 375 kN	375	
5	*Gray stiff hard clayey silt with fine sand – cohesive soil* N60 = 32 ϕ = 8.05 deg Embedded length, D = 28 – (2.5 + 6 + 8 + 4) = 7.5 m $\gamma_{sat} - \gamma_w$ = 10 kN/m^3 Pile diameter, d = 0.6 m c_u = 96.19 KPa D/d = 12.5 α = 0.4 [refer to Figure 9.19 Workshop Manual]		

TABLE 5.3 (*Continued*)
Pile bearing capacity calculation (axial capacity)

Layer	Calculation	Pile capacity (kN)	Reference
	$As = 3.14 \times 0.6 \times 7.5 = 14.13\ m^2$		
	$At = (3.14 \times 0.6^2) / 4 = 0.28\ m^2$		
	$Qu = (\alpha\ c_u\ As) + (Nc\ c_u\ At)$		
	$= (0.4 \times 96.19 \times 14.13) + (9 \times 96.19 \times 0.28)$	786	
	$= 544 + 242 = 786$ kN		
	Sum =	2433	kN
	Factor of safety (FOS) =	2.5	
	Pile capacity (axial) =	**973**	kN

Axial load by bearing at pile tip = 242 kN (10% of total capacity)
Friction load by pile shaft = 2433–242 = 2191 kN (90% of total capacity)

5.1.1.5 Lateral capacity

The lateral capacity should be determined by computer analysis using the pile as a stick model and soil subgrade modulus as spring support. The deflection should be limited from 6 to 12 mm to get allowable lateral load. However, actual lateral load capacity should be established by field load test.

Let us use Brom's method for lateral load capacity determination. [Workshop manual Volume I, Sec – 9.7.3.2]

Step 1

Type of soil within critical depth below ground surface, i.e., about 4 to 5 pile diameter/width (b).

Here, b = 600 mm 4.5 b = 2.7 m.
Depth of layer 1 is 2.52 m, so it falls within critical depth.
Type of soil = **cohesive soil**.

Step 2

Determine the coefficient of horizontal subgrade reaction, Kh, within the critical depth for cohesive or cohesionless soil.

a. *Cohesive soils:* $Kh = (n_1\ n_2\ 80\ q_u) / b$
where q_u *= unconfined compressive strength (kPa)*
b = width or diameter of the pile (m)
n_1, n_2 *= empirical coefficients taken from Table 9.10 Values of coefficients n1 and n2 for cohesive soils (Workshop manual);*

Choose K_h *from Table 9.11 Values of Kh for cohesionless soils in workshop manual (determined by Terzaghi)*
In this example, q_u = 96.44 KPa (=2cu)
n1 = 0.36 n2 = 1.15 and b = 0.6 m (pile dia)
$K_h = 0.36 \times 1.15 \times 80 \times 96.44 / 0.6 = 5323\ kN/m^3$

Step 3

Adjust K_h for loading and soil conditions

a. *Earthquake loading in cohesionless soil:*
   1. *K_h = 0.5 K_h obtained from Step 2 for medium to dense soil*
   2. *K_h = 0.25 Kh obtained from Step 2 for loose soil*

b. *Static load resulting from soil creep in cohesive soil:*
   1. *Soft and very soft normally consolidated clay, K_h = 0.3 to 0.2 K_h from Step 2*
   2. *Stiff to very stiff clay – K_h = 0.25 to 0.5 K_h from Step 2*

Hence, the adjusted value of K_h = 0.25, Kh = 0.25 × 5323 = **1331** kN/m^3

Step 4

Pile parameters:

a. *Modulus of elasticity, E in MPa*
b. *Moment of inertia, I in m4*
c. *Section modulus, S (m3) about an axis perpendicular to the load plane*
d. *Yield stress of pile material steel, fy*
e. *Embedded pile length, D in m – [See Figure 9.27 to 9.29 and Figure 9.31–9.33 Workshop manual]*
f. *Diameter or width, b in m*
g. *Eccentricity of applied load e_c for free-headed piles – i.e., vertical distance between ground surface and lateral load (m)*
h. *Dimensionless shape factor Cs – for steel piles only*
   Use 1.3 for circular pile; 1.1 for H section piles when the applied lateral load is in the direction of the pile's maximum resisting moment (normal to the pile flange); Use 1.5 for H section piles when the applied lateral load is in the direction of pile's minimum resisting moment (parallel to the pile flanges).
i. *My, resisting moment of the pile.*
   My = Cs.fy.S kNm for steel piles
   My = fc′. S kN m for concrete piles

In this example,

Pile diameter or width, b = 0.6 m,   fck = 25 Mpa, fc′ = 30 MPa

a. Ec = 5000√fck = 25000 N/mm^2 (= 25000000 kN/m^2)
b. I = (3.14 × 0.6^4) / 64 = 0.0063585 m^4
c. S = I / (b/2) = 0.021 m^3
d. fy = 415 MPa;   fc′ = 30 MPa
e. D = 28 m
f. b = 0.6 m
g. e_c = 0.15 m (assumed)
h. Cs = Not applicable
i. My = fc′. S = 30 × 1000 × 0.021 = **630** kNm

Step 5

Determine βh for cohesive soil or η for cohesionless soil

$\beta_h = \sqrt[4]{(K_h\, b / 4EI)}$ *for cohesive soil, or*
$\eta = \sqrt[5]{(K_h / EI)}$ *for cohesionless soil*
so, $\beta_h = \sqrt[4]{1331 \times 0.6 / (4 \times 25000000 \times 0.0063585)} = 0.19$

Step 6

Determine the dimensionless length factor

$\beta_h D$ *for cohesive soil*
ηD *for cohesionless soil*

Step 7 Determine if the pile is long or short

a. *Cohesive soil-* $\beta_h D > 2.25$ *(long pile) and* $\beta_h D < 2.25$ *(short pile)*
(Note: It is suggested that for βhD values between 2 and 2.5, both long and short pile criteria should be considered in Step 9, and then the smaller value should be used.)

Here, $\beta_h D = 0.19 \times 28 = 5.32$, so it is long pile

b. *Cohesionless soil:* $\eta D > 4$ *(long pile)*
$\eta D < 2$ *(short pile)*
$2 < \eta D < 4$ *(intermediate pile)*

In this example, the pile is a long pile in cohesive soil.

Step 8

Determine other soil parameters over the embedded length of piles.

a. *Rankine passive pressure coefficient for cohesionless soil, Kp*
$Kp = \tan^2 (45 + \phi / 2)$, *where* ϕ *= angle of internal friction*
b. *Average effective unit weight of soil,* γ' *(kN/m³)*
c. *Cohesion,* c_u *(kPa)*
c_u *= Half the unconfined compressive strength,* q_u

In this example, $\phi = 6.6$ (Layer 1)
$\gamma' = 5.16$ kN/m³
$c_u = 48.22$ kPa

Step 9 Determine the ultimate lateral load for a single pile, Qu

See Figure 9.27 to 9.30 in Workshop manual

a. *Short free- or fixed-headed pile in cohesive soil*
Using D/b (and e_c / b *for the free-headed case), in Figure 9.27 Ultimate lateral load capacity of long piles in cohesionless soils – workshop manual, select the corresponding value of* $Qu / c_u.b2$*, and solve for Qu (kN)*

b. *Long free- or fixed-headed pile in cohesive soil*
Using My /c_u.b^3 (and e_c /b for the free-headed case), in Figure 9.28 Ultimate lateral load capacity of long piles in cohesionless soils – workshop manual, select the corresponding value of Qu/c_u b^2 and solve for Qu (kN).

c. *Short free- or fixed-headed pile in cohesionless soil*
Using D/b (and e_c /D for the free-headed case) in Figure 9.30 Ultimate lateral load capacity of long piles in cohesionless soils – workshop manual, select the corresponding value of Qu / Kpb^3/ g and solve for Qu (kN).

d. *Long free- or fixed-headed pile in cohesionless soil*
Using My/b^4 γ Kp (and e_c/b for the free-headed case) in Figure 9.30 Ultimate lateral load capacity of long piles in cohesionless soils – workshop manual, select the corresponding value of Qu/Kp b^3 γ and solve for Qu (kN).

e. *Intermediate free- or fixed-headed pile in cohesionless soil*
Calculate Qu for both short (Step 9c) and long (Step 9d) piles and use the smaller value.

In this workout example, the pile is long fixed-headed pile in cohesive soil at top layer.

Using the curves in *Figure 9.28* Ultimate lateral load capacity of long piles in cohesive soils – workshop manual.

$My/c_u\, b^3 = 630 / (48.22 \times 0.6^3) = 60$
$e_c/b = 0.15 / 0.6 = 0.25$
$Qu/c_u\, b^2 = 35$
For $My = 630$ kNm $e_c = 0.15$ m $c_u = 48.22$ kPa $b = 0.6$ m
$Qu = 35 \times 48.22 \times 0.6^2 = 608$ kN

Step 10 Calculate the maximum allowable working load for a single pile, Qm

$Qm = Qu/2$ $Qm = 608 / 2 = 304$ kN

Step 11

a. *Free- or fixed-headed pile in cohesive soil*
Using β_hD (and e_c/D for the free-headed case), enter Figure 9.32 (Workshop manual), select the corresponding value of y K_h b D / Qa and solve for Qa (kN) or y (m).

b. *Free- or fixed-headed pile in cohesionless soil*
Using ηD (and e_c/D for the free-headed case), enter Figure 9.33 (Workshop manual), select the corresponding value of y $(EI)^{3/5}$ $K_h^{2/5}$/Qa D and solve for Qa (kN) or y (m).

For this example, let us assume the following:

Lateral deflection at the pile head, $y = 10$ mm
$K_h = 1331$ kN/m^3 $b = 0.6$ m depth, $D = 28$ m and
$\beta_h D = 5.32$
From Figure 9.32 in workshop manual,
$y K_h b D / Qa = 4.8$ and
$Qa = (0.01 \times 1331 \times 0.6 \times 28) / 4.8 =$ **47** kN

Step 12 Compare Qa to Qm

a. *if Qa > Qm, use Qm and calculate ym (Step 11)*
b. *if Qa < Qm, use Qa and y*
c. *if Qa and y are not given, use Qm and ym.*

Qm = 304 kN
Qa = 47 kN

In this example, the estimated lateral shear Qa = **47** kN for 10 mm deflection.

5.1.1.6 Uplift capacity

1. Uplift capacity = 1/3 of ultimate shaft resistance = 2433 / 3 = 811 kN
2. Uplift capacity = 70 % of shaft resistance = $0.7 \times 973 = 681$ kN
3. As per Tomlinson, pull out capacity, T = 50% ultimate skin resistance + weight of pile. (It is a conservative approach; 50% loss is generally advised for short pile.)

   $T = 0.5 \times (2433 - 242) / 2.5 + (3.14 \times 0.6^2 / 4) \times 24 \times 28 = 677$ kN
   FOS = 2.5

Pile capacity (uplift) = 677 kN

5.1.1.7 Summary of pile capacity

Pile diameter = 600 mm
Length of the pile = 28 M
Pile capacities for strength design and settlement analysis:

Axial compression = 900 kN [for 15 mm foundation settlement limit]
Lateral = 47 kN
Uplift = 450 kN

[Notes: Axial capacities should be checked by performing settlement analysis of pile groups and applying group efficiency factors. The allowable design capacity of pile and its founding level should satisfy the permissible settlement limit.

There are sites where piles carry negative friction due to imposed filling layer at top and subsequent consolidations of soft clay at bottom layers. Hence, the allowable *capacity will be further reduced by deducting the calculated value of the negative friction. The method of calculating the negative friction is given in section 18.8 in Bowels.*

[However, the actual capacity shall be determined after the pile load test at field.]

5.1.1.8 Structural strength design of the pile

Let us consider the following design capacity for the pile:

Axial = 900 kN
Lateral = 47 kN
Uplift = 450 kN
Moment (M) at top = 630 kNm

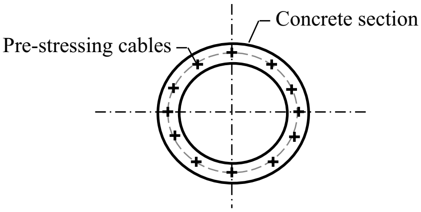

FIGURE 5.4 Hollow circular section

The following load combinations are to be used for reinforcement and strength design:

1. Axial + lateral
2. Uplift + lateral

Strength design should be carried out as per standards provided in PCI design Handbook for Precast and Pre-stressed Concrete, ACI 318 or IS code 1343: 2012. The Precast pre-stressed piles are also available as factory-made product as per manufacturer's standard design for wide range of load. Users can select the appropriate size of the pile based on their requirement. These piles may be hollow sections or solid sections – rectangular or square. The standard drawing details of precast pre-stressed piles are available in Section 6.

5.1.1.9 Settlement analysis of pile group

The following calculation shows how to calculate the settlement of pile groups. In this analysis, a group of nine piles has been considered.

TABLE 5.4
Soil parameters for settlement calculation

Layer	Description	Depth (m)	N60	Cc	e_0	γ_{sat}	Remarks
1	Gray soft silty clay	5	5	0.262	1.062	15.16	
2	Gray medium-stiff sandy silt	6	16			18.5	
3	Gray medium-dense silty sand with clay	8	22			18.5	
4	Gray medium-dense silty and fine sand with clay	4	25	0.15	0.9	20	
5	Gray stiff hard clayey silt with fine sand	17	32	0.2	0.9	20	Founding level
6	Gray stiff hard clayey silt with fine sand	15	32	0.2	0.9	20	

FGL = 0.0 M EGL = 0.0M GWL = 0.0 M
Pile cut-off level = –2.5 M
Pile diameter = 0.6 M length of pile, Lp = 28 M
Pile group = 9G (nine pile group) number of piles = 9
Group efficiency = 0.7 pile capacity = 900 kN (Axial)
Total load, P = 9 × 0.7 × 900 = 5670 kN
Pile cap size: length, L = 4.35 M width, B = 4.35 M

Sketch

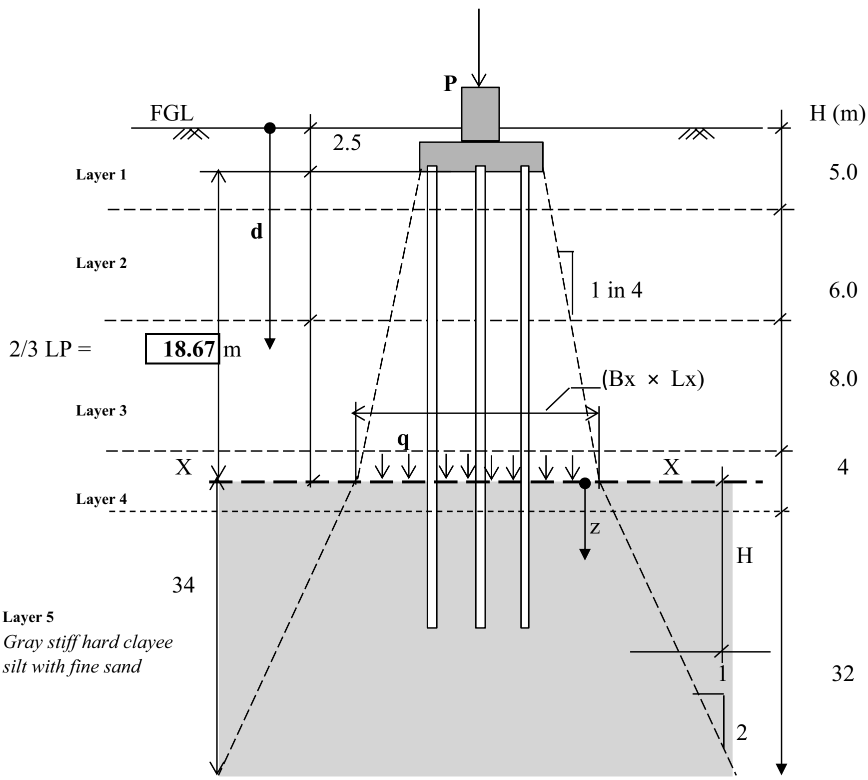

FIGURE 5.5 Pressure distribution below equivalent footing for pile group

Total settlement = SI + SII
Effective layer of settlement = Heff

[The depth at which the pressure increment is less than 10% of overburden pressure will provide the total thickness (Heff) of soil to be evaluated in the settlement computation. (Reference: CL 9.8.2.3 in Workshop Manual Vol I)]

Initial Settlement, S_I

S_I = elastic compression of pile = Qu. L / (A. Ec)
Ec = 25000 N/mm^2 A = (3.14 × 0.6^2) / 4 = 0.28 m^2
Qu = Qpa + αs. Qfa
where Qpa = axial load by bearing at the pile tip = 10 % × 900 = 90 kN
Qfa = axial load by shaft friction = 90 % × 900 = 810 kN
αs = 0.5 for uniform distribution of skin friction
Qu = 90 + 0.5 × 810 = 495 kN
Lp = 28 m S_I = 49500 × 28000 / (282600 × 25000) = **2.0** mm.

Consolidation settlement of cohesive layers below X-X, S_{II}:

$\Delta H = H\,[(Cc / (1 + e_0)) \times \log (p_0 + \Delta p) / p_0]$
H = Σ h = original thickness of stratum in mm
Cc = compression index e_0 = initial void ratio
p_0 = effective overburden pressure at the midpoint of the compressible stratum prior to pressure increase in kPa
Δp = average change in pressure in the compressible stratum in kPa
ΔH= total settlement in mm
$p_0 = q + [\gamma_{sat}.d - \gamma w. (GWL- d)]$
z = depth of point, where p_0 is taken, below X-X
Δp = P/A; A = area at the middle of the segment
q = surcharge above X-X = 30 kN/mm^2
Bx = Lx = 4.35 + 0.5 × 18.67 = 13.7 m area at X-X = 13.7 × 13.7 = 188 m^2
P = 5670 kN
p_0 = 30 + (15.16–10) × 5 + (18.5–10) × 6 + (18.5–10) × 8 = 175 kN/m^2
A = (13.7 + 0.5) × (13.7 + 0.5) = 202 m^2
Δp = 5670 / 202 = 28.07 kN/m^2
$\Delta H = 1 \times [(0.15 / (1 + 0.9) \times \text{Log}_{10} \{(175 + 28.07) / 175\}] \times 1000 = 5.101$ mm.

Computation of consolidation settlement over 25 years [Bowel – 2.10.1]

From theory of consolidation, we get, T = cv. t / H^2
where t = time of consolidation = 25 years

TABLE 5.5
Settlement in layers below X-X

Segment	h m	z m	$\gamma_{sat} - \gamma_w$ kN/m^3	p_0 kN/m^2	A m^2	Δp kN/m^2	Cc	e_0	ΔH mm	Total Deflection	p_0/Δp [*]
1	1	0.5	10	175	202	28.07	0.15	0.9	5.101		**0.14**
2	1	1.5	10	185	231	24.59	0.2	0.9	5.705	**10.81**	0.11
3	1	2.5	10	195	262	21.65	0.2	0.9	4.812		0.08
4	1	3.5	10	205	295	19.20	0.2	0.9	4.093		0.07

[*] The depth at which the pressure increment is less than 10% of overburden pressure will provide the total thickness of soil to be evaluated in the settlement computation.

H = effective thickness of the soil layer = 200 cm
Cv = coefficient of consolidation = 0.00035 cm^2 /sec
T = a dimensionless time factor
T = cv. t /H^2 = 6.899
Now, amount of consolidation, U = √ (4T / p) for 0 < T <= 0.197
Amount of consolidation, U = 1 – (8 / p^2) × $e^{-p2.\ T/4}$ for T > 0.197
So, U = 100%
Total consolidation settlement, S_{II} = 1 × 10.81 = 10.81 mm
Total settlement, $S_I + S_{II}$ = 1.96 + 10.81 = 12.8 mm < 15 mm (generally allowable for foundations.)

Conclusion: The pile capacities calculated above are based on field tests and laboratory data obtained by geotechnical investigation. The actual capacity should be established by pile load tests in the field.

5.1.2 Driven pile in sand

(Reference: Pile Design and Construction Practice – M. J. Tomlinson, Foundation Analysis and Design by J. E. Bowels and Design and Construction of Driven Pile Foundation – Workshop Manual Vol I – US Dept of Transportation (FHWA HI 97–013); ACI 318–19 Building Code Requirements for Structural Concrete.)

Pile Type: Precast pre-stressed concrete hollow pile/cast-in-situ pile using steel casing

Pile diameter, d = 400 mm
Pile length, Lp = 22 M
Pile cut-off level = –3 M
finish-grade level, FGL = 0.0M
groundwater level, GWL = 0.0 M

5.1.2.1 Sketch

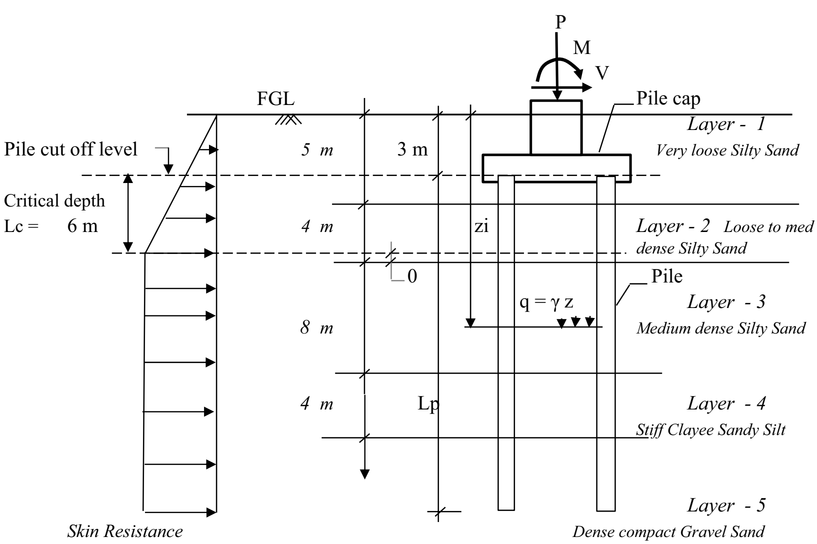

FIGURE 5.6 Piles in sand

5.1.2.2 Soil parameters

TABLE 5.6
Properties of soil stratum from bore hole data

Layer mark	Description	Thickness (D) in meter	SPT value (N60)
1	Very loose silty sand	5	3
2	Loose to medium-dense silty sand	4	8
3	Medium-dense silty sand	8	17
4	Stiff clayey sandy silt	4	51
5	Gravel sand dense compact	19	60

TABLE 5.7
Properties from laboratory tests

	γs	Cu	ϕ	
Layer	kN/m³	kPa	deg	Nq
1	19.6	10	20	10
2	19.5	10	20	10
3	22.6	9	28	24
4	22.7	9	29	27
5	22.7	0	29	27

5.1.2.3 Pile capacity calculation

a. Axial compression:

Cohesionless Soil

End bearing $Qb = Nq\ \sigma_{0v}\ Ab$,

where Nq = bearing capacity factor

σ_{0v} = effective overburden pressure = γ zi

Ab = cross-sectional area at the pile toe

Skin friction $Qs = \Sigma\ (\gamma s\ zi\ K \tan \phi\ As)$

where As = surface area of pile shaft

γs = effective soil density

zi = effective depth of the mid layer below FGL

ϕ = angle of shearing resistance of soil

K = coefficient of earth pressure at rest (K_0)

Note: Critical depth for skin friction = 15 times pile diameter for loose to medium sand and 15 to 20 times pile diameter for medium to stiff layers of sand. Skin friction will be constant below critical depth.

5.1.2.3 Axial capacity

TABLE 5.8
Axial compression load capacity calculation

Layer	Calculation	Pile capacity (kN)
1	*Very loose silty sand*	
	N60 = 3	
	Embedded length, D = 5 − 3 = 2.0 m	
	$\gamma_s = \gamma_{sat} - \gamma_w = 9.6$ kN/m^3	
	$\phi = 20$ deg	
	Pile diameter, d = 0.4 m	
	$c_u = 10$ kPa	
	As = 3.14 × 0.4 × 2 = 2.51 m^2	
	tan ϕ = 0.364; sin ϕ = 0.342; K = 1 − sin ϕ = 0.658	
	zi = 5 − 0.5 × 2 = 4 m	
	Qs = Σ (γs zi K tan ϕ As)	
	= 9.6 × 4 × 0.658 × 0.364 × 2.512 = 23 kN	23
2	*Loose to medium-dense silty sand*	
	N60 = 8	
	Layer thickness = 4.0 m	
	Critical depth, Lc = 15 × 0.4 = 6 m	
	Thickness of Layer 2 above Lc = 3 + 6 − 5 = 4.0 m	
	$\gamma_s = \gamma_{sat} - \gamma_w = 9.5$ kN/m^3	
	$\phi = 20$ deg	
	Pile diameter, d = 0.4 m	
	$c_u = 10$ kPa	
	As = 3.14 × 0.4 × 4 = 5.024 m^2	
	tan ϕ = 0.364; sin ϕ = 0.342; K = 1 − sin ϕ = 0.658	
	zi = 5 + 0.5 × 4 = 7 m	
	γs. zi = 9.5 × 7 = 66.5 kN/m^2	
	Qs = Σ (γs zi K tan ϕ As)	
	= 9.5 × 7 × 0.658 × 0.364 × 5.024 = 80 kN	80
3	*Medium-dense silty sand*	
	N60 = 17	
	Layer thickness = 8.0 m	
	$\gamma_s = \gamma_{sat} - \gamma_w = 12.6$ kN/m^3	
	$\phi = 28$ deg	
	Pile diameter, d = 0.4 m	
	$c_u = 9$ kPa	
	As = 3.14 × 0.4 × 8 = 10.05 m^2	
	tan ϕ = 0.532; sin ϕ = 0.469; K = 1 − sin ϕ = 0.531	
	Skin friction = γs. zi = 9.5 × 7 = 66.5 kN/m^2 (constant below critical depth)	
	Qs = Σ (γs zi K tan ϕ As)	
	= 66.5 × 0.531 × 0.532 × 10.05 = 189 kN	189

(*Continued*)

TABLE 5.8 (*Continued*)
Axial compression load capacity calculation

Layer	Calculation	Pile capacity (kN)
4	*Stiff clayey sandy silt*	
	N60 = 51	
	Layer thickness = 4.0 m	
	$\gamma_s = \gamma_{sat} - \gamma_w = 12.7$ kN/m^3	
	$\phi = 29$ deg	
	Pile diameter, d = 0.4 m	
	$c_u = 9$ kPa	
	As = 3.14 × 0.4 × 4 = 5.024 m^2	
	tan ϕ = 0.554; sin ϕ = 0.485; K = 1 − sin ϕ = 0.515	
	Skin friction = γs. zi = 9.5 × 7 = 66.5 kN/m^2 (constant)	
	Qs = Σ (γs zi K tan ϕ As)	
	= 66.5 × 0.515 × 0.554 × 5.024 = 95 kN	95
5	*Dense compact gravel sand*	
	N60 = 60	
	Effective thickness = 22 − 2 − 4 − 8 − 4 = 4.0 m	
	$\gamma_s = \gamma_{sat} - \gamma_w = 12.7$ kN/m^3	
	$\phi = 29$ deg	
	Pile diameter, d = 0.4 m	
	$c_u = 0$ kPa	
	As = 3.14 × 0.4 × 4 = 5.024 m^2	
	Ab = (3.14 × 0.4^2) / 4 = 0.126 m^2	
	tan ϕ = 0.554; sin ϕ = 0.485; K = 1 − sin ϕ = 0.515	
	Skin friction = γs. zi = 9.5 × 7 = 66.5 kN/m^2 (constant)	
	Nq = 27	
	[refer to Figure 1 in IS:2911 bearing capacity factor (Nq) vs angle of internal friction (ϕ)]	
	Qs = Σ (γs zi K tan ϕ As)	95
	= 66.5 × 0.515 × 0.554 × 5.024 = 95 kN	
	σ_{0v} = 5 × 9.6 + 4 × 9.5 + 8 × 12.6 + 4 × 12.7 + 4 × 12.7 = 288 kN/m^2	
	End bearing capacity = Qb = Nq σ_{0v} Ab	
	= 27 × 288 × 0.126 = 977 kN	977
	Sum	1460
	Factor of safety, FOS =	2.5
	Pile capacity (axial) =	**584**

Axial load by bearing at the pile tip = 977 kN (67%)

Friction load by pile shaft = 1460−977 = 483 kN (33%)

5.1.2.4 Lateral capacity

The lateral capacity is usually determined by computer analysis considering the pile as a stick model and soil subgrade modulus as spring support. The deflection should be limited from 7 to 12 mm to get allowable lateral load. However, the actual lateral load capacity should then be established by field load tests.

In the absence of computer analysis, the following method can be used to get the lateral load at the pile head. [Refer to IS 2911 (Part 1/Sec 2): 2010 – ANNEX – C].

The long flexible pile is treated as a cantilever fixed at some depth below the ground level.

The lateral resistance for granular cohesionless soil and normally consolidated clays which have varying soil modulus is modeled according to the equation:

$$p / y = \eta_h z$$

where p = lateral soil reaction per unit length of pile at depth z below the ground level

y = lateral pile deflection

η_h = modulus of subgrade reaction (Table 5.9 for recommended values)

TABLE 5.9
Modulus of subgrade reaction for granular soils, ηh in kN/m³

SL No	Soil type	N (blows/30cm)	Range of η_h kN/m³ × 10³ Dry	Range of η_h kN/m³ × 10³ Submerged
1	Very loose sand	0 – 4	<0.4	< 0.2
2	Loose sand	4 – 10	0.4 – 2.5	0.2 – 1.4
3	Medium sand	10 – 35	2.5 – 7.5	1.4 – 5
4	Dense sand	> 35	7.5 – 20	5 – 12

Note: η_h values may be interpolated for intermediate standard penetration value.

For piles in sand stiffness factor T, in m = $\sqrt[5]{(EI / \eta_h)}$
$E_c = 5000 \sqrt{fck}$ N/mm² = 25000 MPa (= E) fck = 25 MPa
$I = (3.14 \times 0.4^4) / 64 = 0.001256$ m⁴
$\eta_h = 1.8 \times 10^3$ kN/m³
$T = \sqrt[5]{(25000 \times 0.001256 / 1.8 \times 10^3)} = 0.445$ m.
In our case, L1 = 0 m So, L1 / T = 0

From Figure 2 Curves for depth of fixity (IS: 2911),
we get Lf / T = 2.2
Lf = 2.2 × 0.445 = 0.98 m.

Now, pile head deflection, y, shall be determined using any of the following equations:

Deflection, y = (Q Lf^3 / 3 Ec I) x 10^3 mm for free-headed pile
Deflection, y = (Q Lf^3 / 12 Ec I) x 10^3 mm for fixed-headed pile
where y = pile head deflection in mm Ec = 25000 MPa
Q = lateral load in kN I = 0.001256 m^4

In our case, y = (Q Lf^3/12 EI) x 10^3 = 0.98^3 / (12 × 25000 × 0.001256) × Q × 1000
= 2.5 Q mm

TABLE 5.10
Y* vs *Q

Y	Q
Mm	kN
5	2.00
6	2.40
7	2.80
8	3.20
10	4.00
12	4.80

Let us consider the maximum allowable deflection is 12mm. So, for y = 12 mm, Q = 4.80 kN

Moment at the top of the pile, M = m. Q. Lf

Reduction factor, m = 0.82 [refer to Figure 3 Curves for moment reduction factor of IS 2911]

For L_1 / T = 0
M = 0.83 × 4.8 × 0.98 = **3.86** kNm

Pile capacity (lateral) = **4.80** kN

5.1.2.5 Axial tension or uplift capacity

As per Tomlinson, pull out capacity, T = 50% ultimate skin resistance + weight of the pile
T = 0.5 × (1460–977) / 2.5 + (3.14 × 0.4^2 / 4) × 25 × 22 = 166 kN
F.O.S = 2.5
Pile capacity (uplift) = **166** kN.

5.1.2.6 Design capacity of pile

Let us consider the following design capacity for the pile:

Axial compression = 550 [for allowable settlement up to 6 mm]
Lateral = 4.80 kN
Uplift = 150 kN

The above data are selected as the design capacity of the pile. The strength design and settlement analysis will be done using the design capacity.

There are sites where piles carry negative skin friction due to the imposed filling layer at the top and subsequent consolidation of soft clay at the bottom layers. Hence, the allowable capacity will be further reduced by deducting the calculated value of the negative friction. The method of calculating negative friction is illustrated in section 18.8 in Bowels.

However, the actual capacity shall be confirmed by field tests on the test pile.

5.1.2.7 Settlement analysis of pile group

The following calculation shows how to calculate the settlement of pile groups. In this analysis, a group of five piles has been considered.

TABLE 5.11
Soil parameters for settlement calculation

Layer	Description	Depth (m)	N60	γ_{sat} kN/m³	Remarks
1	Very loose silty sand	5	3	19.6	
2	Loose to medium-dense silty sand	4	8	19.5	
3	Medium-dense silty sand	8	17	22.6	
4	Stiff clayey sandy silt	4	51	22.7	
5	Gravel sand dense compact	19	60	22.7	Founding level
6	Gravel sand dense compact	10	60	22.7	

FGL = 0.0 M EGL = 0.0M GWL = 0.0 M
Pile cut-off level = (–) 3.0 M
Pile diameter = 0.4 M length of pile, Lp = 22 M
Pile group = 5G (five pile group) number of piles = 5
Group efficiency = 1 pile capacity = 550 kN (Axial)
Total load, P = 5 × 1 × 550 = 2 750 kN
Pile cap size: length, L = 2 M width, B = 2 M

Total settlement, $S = S_I + S_{II}$
Effective layer of settlement = Heff

[The depth at which the pressure increment is less than 10% of overburden pressure will provide the total thickness (H) of soil to be evaluated in the settlement computation.]

a) Initial settlement, S_I (CL 9.8.2.1 workshop manual)

S_I = elastic compression of pile = Qu. L / (A. Ec)
Ec = 25000 N/mm² A = $(3.14 \times 0.4^2) / 4 = 0.13$ m²
Qu = Qpa + αs. Qfa

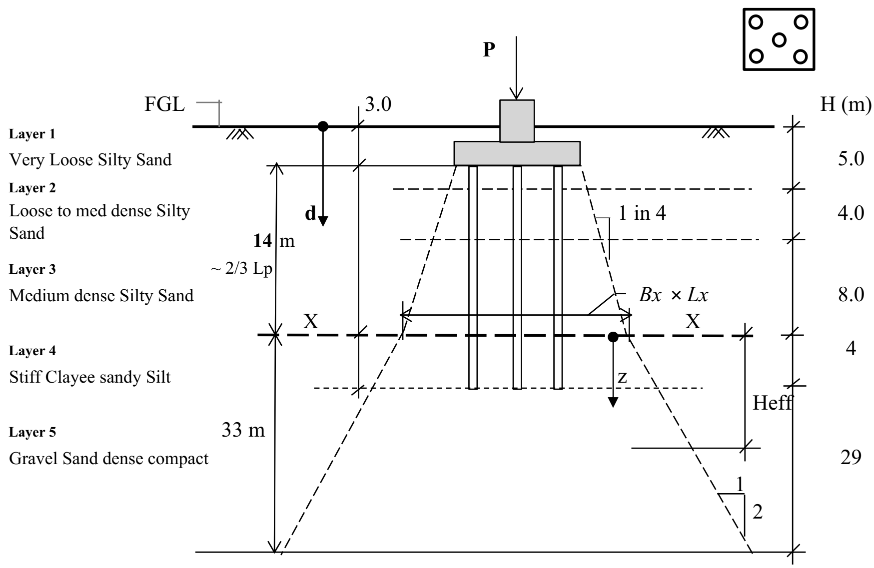

FIGURE 5.7 Pressure distributions below pile group for settlement analysis

where Qpa = axial load by bearing at pile tip = 67% × 550 = 369 kN

Qfa = axial load by shaft friction = 33% × 550 = 182 kN

αs = 0.5 for uniform distribution of skin friction

Qu = 369 + 0.5 × 182 = 460 kN

Lp = 22 m S_I = 460000 × 22000 / (125600 × 25000) = 3.2 mm

b) Settlement of pile group in the sand layer, S_{II}:

[refer to Design of Pile foundation workshop manual Vol 1- Section 9.8.2.4]

$\Delta H = H [(1/C') \log (p_0 + \Delta p) / p_0]$

Heff = 6.0 m

$p_0 = q + [\gamma sat.\ d - \gamma w.\ (GWL- d)]$

z = depth of point, where p_0 is taken, below X-X

Δp = P/A A = effective area at mid-section

C′ = bearing capacity index value for SPT 55 average (assumed 200) – see Figure 9.45 Value of bearing capacity index C′ for granular soil (workshop manual)

P = 2750 kN. Bx = Lx = 2 + 0.5 × 14 = 9 m Area at X-X = 9 × 9 = 81 m^2

q = surcharge above X-X due to column load = 2750/81 = 34 kN/m^2

5.1.2.8 Structural strength design of pile

Precast pre-stressed concrete piles may be used for driven piles to save time and cost. Strength design should be done as per guidelines provided in the PCI Design Handbook for Precast and Pre-stressed Concrete, ACI 318, or IS Code 1343: 2012.

TABLE 5.12
Calculation of settlement s_{II} in sand layers

Segment	h (m)	z (m)	$\gamma_{sat} - \gamma_w$ (kN/m³)	p_0 (kN/m²)	A (m²)	Δp (kN/m²)	Cc	ΔH (mm)	Total Deflection	$p_0/\Delta p$ [*]
1	1.5	0.75	12.7	221	95	28.93	200	0.40	**0.89**	0.30
2	1.5	2.25	12.7	240	127	21.73	200	0.28		0.17
3	1.5	3.75	12.7	259	163	16.92	200	0.21		0.10
4	1.5	5.25	12.7	278	203	13.54	200	0.15		0.07
5	1.5	6.75	12.7	297	248	11.09	200	0.12		0.04
6	1.5	8.25	12.7	316	298	9.24	200	0.09		0.03
7	1.5	9.75	12.7	335	352	7.82	200	0.08		0.02

[*] The depth at which the pressure increment is less than 10% of overburden pressure will provide the total thickness of soil to be evaluated in the settlement computation.
Total settlement, $S = S_I + S_{II} = 3.2 + 0.89 = 4.11 < 6$ mm

These piles are also available as ready-made units from the manufacturer's shop as per the manufacturer's standard design for specified loads. These piles may be hollow sections or solid sections – rectangular or square. The standard drawing details of precast pre-stressed piles are available in Section 6.

Driven cast-in-situ piles may also be used for cohesionless or cohesive soil in layered formation. This type of pile is installed by driving a casing of uniform diameter up to the specified depth and then subsequently withdrawing it when filling the hole with concrete. To displace the soil, the casing is provided with a cast iron shoe at the bottom end. When the casing is left permanently, it is termed a cased pile, and when the casing is taken out, it is termed an uncased pile. Alternately, a rig-mounted auger drill is used to dig a hole in the ground. The hole is protected by filling it with bentonite slurry.

In both cases, a reinforcement cage is lowered inside the hole before pouring concrete.

5.1.2.9 Structural strength design of driven cast-in-situ pile

- ACI 318–19 (SI unit; stress in Mpa)

Design load (unfactored):

Axial, P = 550 kN; lateral, V = 4.80 kN; uplift, T = 150 kN
Moment at top, M = 3.6 kNm

The following load combinations are to be used for reinforcement and strength design:

1. Axial + lateral
2. Uplift + lateral

Concrete grade = M25 $f_c' = 25\ N/mm^2$ (3600 psi)
$f_y = 415\ N/mm^2$ (60000 psi)
Clear cover = 75 mm Pile dia, d = 400 mm $\gamma = 0.6$
$A_g = 125600\ mm^2$ (195 $inch^2$)
$E_c = 5700\ \sqrt{f_c'} = 28500\ mm^2$
Here, the value of moment at top is less than 5 % of pile diameter.
Hence, the pile is designed as per the allowable stress method (Table 13.4.2.1-ACI 318).
(5% × 400 = 20 kNm > M = 3.6 kNm)

5.1.2.10 Pile reinforcement

Longitudinal bars: 4 nos. 16 mm diameter + 4 nos. 12 mm diameter
Spiral bars – stirrups: 8 mm diameter @ 150 mm C/C
Ast = 1256 mm^2 > = minimum reinforcement
p = (1256 / 125600) × 100 = 1.00 %

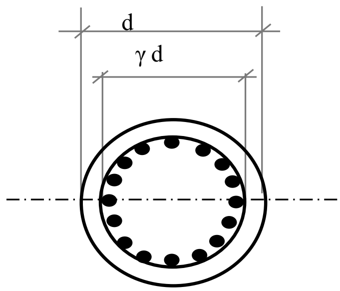

FIGURE 5.8 Pile reinforcement

Check for axial load

P_0 = maximum allowable compression strength
= 0.3 fc′ Ag +0.4 fy Ast
= 0.3 × 25 × 125600 + 0.4 × 415 × 1256 = 1150 kN
Pmax = 0.8 P0 = 920 kN $\phi = 0.55$
P_{allow} = 920 × 0.55 = 506 kN > P; safe.

Check for tensile strength

Pn t max = fy. Ast = 415 × 1256 / 1000 = 521 kN
Uplift = T = Tu. ϕ = 521 × 0.65 = 339 kN > T safe.

Notes:

1. *Reinforcement limits:* *ACI 318–19 (10.6.1.1)*
   Minimum reinforcement = 0.01 Ag =1256 mm^2
   Maximum reinforcement = 0.08 Ag =10048 mm^2
2. *Geometric assumptions for calculation:* *ACI 318–19 (22.5.2.2)*
   d = distance from extreme comp. fiber to centroid of longitudinal tension reinforcement = 0.8 time diameter of pile
   bw = effective width of comp. flange = Diameter of pile
3. *The pile can be designed using the allowable stress design method if it is laterally supported for its entire height and the moment is less than 5% of the diameter or width. Maximum allowable axial load, Pa = 0.3 fc′ Ag + 0.4 fy As per ACI 318–19 (13.4.2.1)*

Or,

The pile shall be designed as a column (Cl 10.5) for tension, shear and combined axial force and moment. The strength reduction factor f will be 0.65 as per 21.2.1. The maximum axial compression strength, Pn = 0.8 Po; where Po = 0.85 fc′ (Ag – Ast) + fy Ast.

The strength reduction factor f will be 0.55 as per Table 13.4.3.2 for axial load without moment.

5.1.3 Bore pile in layered soil

(Reference: Pile Design and Construction Practice – M. J. Tomlinson, Foundation Analysis and Design by J. E. Bowels and IS: 2911 (Part I/Sec 2) – 2010)

Pile diameter = 600 mm	FGL = 0.0 M
Pile length, Lp = 28 M	EGL = –1.0 M
Pile cut-off level = – 2.5 M	GWL = 0.0 M (assumed for design)

5.1.3.1 Soil parameters

TABLE 5.13
Description of soil from bore hole data

Layer mark	Description	Thickness in meter	SPT value (N60)
1	Gray soft silty clay	5	5
2	Gray medium-stiff sandy silt	6	16
3	Gray medium-dense silt and sand silt	8	22
4	Gray very stiff silty clay	4	25
5	Gray stiff hard silty clay	17	32

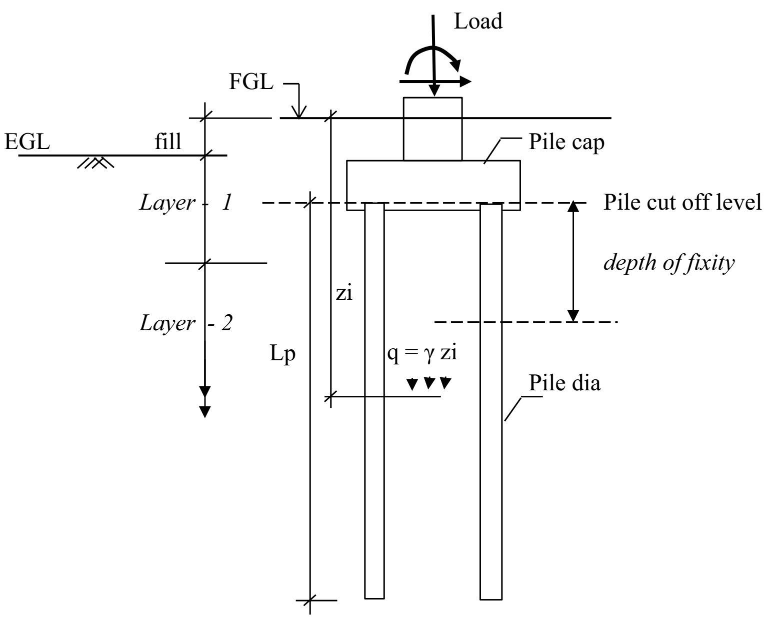

FIGURE 5.9 Bore pile in layered soil

TABLE 5.14
Laboratory data

Layer	γs (kN/m³)	Cu (kPa)	ϕ (deg)
1	15.16	48.22	6.6
2	18.5	51.65	7.32
3	18.5	21.66	29.48
4	20	57.21	8.77
5	20	96.19	8.05

TABLE 5.15
Value of Cu vs α for bore pile

Cu (kN/m²)	α
40	1
50	0.87
60	0.75
70	0.63
80	0.56
90	0.49
100	0.44
110	0.4
120	0.36
130	0.33
140	0.31
150	0.29
160	0.28
170	0.27
180	0.265
190	0.26
200	0.26

5.1.3.2 Pile capacity determination

Cohesive soil

End bearing $Qb = Nc\ p_t\ At$
Skin friction $Qs = \Sigma\ \alpha\ c_u\ As$
Ultimate compression load, $Qu = Qb + Qs$

where $Nc = 9$
p_t = effective overburden pressure at the pile toe
At = pile toe area

α = adhesion factor (clay)
c_u = average cohesion of soil
As = surface area of the pile shaft
γs = effective density of soil
ϕ = angle of shearing resistance
d = embedded length

Cohesionless soil

End bearing $Qb = Nq\ p_t\ At$,
where Nq = bearing capacity factor
p_t = effective overburden pressure at the pile toe
At = pile toe area
Skin friction $Qs = K_0\ p_d \tan\phi\ As_1$
where K_0 = coefficient of earth pressure at rest at depth d
p_d = effective overburden pressure at the center of depth increment, d
As_1 = surface area of the pile shaft (perimeter x embedded depth)
d = embedded length

[Note: The unit skin friction remains at a constant value below a penetration depth of 10 to 20 diameters of pile (Pile Design and Construction Practice by Tomlinson)]

5.1.3.2.1 Axial compression

TABLE 5.16
Axial compression load capacity calculation

Layer	Calculation	Pile Capacity (kN)
1	*Gray soft silty clay* cohesive soil	
	N60 = 5 ϕ = 6.6 deg.	
	Embedded length, d = 5 + 1 − 2.5 = 3.5 m	
	$\gamma_s = \gamma_{sat} - \gamma_w = 5.16\ kN/m^3$	
	Pile diameter, d = 0.6 m	
	c_u = 48.22 kPa; α = 0.89 [From Table 5.15]	
	As = 3.14 × 0.6 × 3.5 = 6.594 m^2	
	Skin friction $Qs = \alpha\ c_u\ As = 0.89 \times 48.22 \times 6.594 = 283$ kN	283
2	*Gray medium-stiff sandy silt* cohesive soil	
	N60 = 16 ϕ = 7.32 deg.	
	Embedded length, d = 6 m	
	$\gamma_s = \gamma_{sat} - \gamma_w = 8.5\ kN/m^3$	
	Pile diameter, d = 0.6 m	
	c_u = 51.65 kPa; α = 0.85	
	As = 3.14 × 0.6 × 6 = 11.3 m^2	
	Skin friction $Qs = \alpha\ c_u\ As = 0.85 \times 51.65 \times 11.3 = 496$ kN	496

(*Continued*)

TABLE 5.16 (*Combined*)
Axial compression load capacity calculation

Layer	Calculation	Pile Capacity (kN)
3	*Gray medium-dense silty and fine sand* cohesionless soil	
	N60 = 22 ϕ = 29.48 deg.	
	Embedded length, d = 8 m	
	[critical depth for skin friction = 20 × 0.6 − 3.5 − 6 = 2.5 m]	
	$\gamma_s = \gamma_{sat} - \gamma_w$ = 8.5 kN/m^3	
	Pile diameter, d = 0.6 m	
	c_u = 21.66 kPa; α = 1 [From Table 5.15]	
	$As = As_1$ = 3.14 × 0.6 × 8 = 15.07 m^2	
	$K_0 = 1 - \sin\phi$ = 1 − 0.492 = 0.508	
	p_d = 5.16 × 5 + 8.5 × 6 + 8.5 × 0.5 × 2.5 = 87 kPa	
	$\tan\phi$ = 0.565	
	Skin friction, $Qs = K_0\, p_d \tan\phi\, As_1$	
	= 0.508 × 0.87 × 0.565 × 15.07 = 376 kN	376
4	*Gray very stiff silty clay* cohesive soil	
	N60 = 25 ϕ = 8.77 deg.	
	Embedded length, d = 4 m	
	$\gamma_s = \gamma_{sat} - \gamma_w$ = 10 kN/m^3	336
	Pile diameter, d = 0.6 m	
	c_u = 57.21 kPa; α = 0.78	
	As = 3.14 × 0.6 × 4 = 7.54 m^2	
	Skin friction $Qs = \alpha\, c_u\, As$ = 0.78 × 57.21 × 7.54 = 336 kN	
5	*Gray stiff hard silty clay* cohesive soil	
	Founding layer	
	N60 = 32 ϕ = 8.05 deg.	
	Embedded length, d = 28 − 3.5 − 6 − 8 − 4 = 6.5 m	
	$\gamma_s = \gamma_{sat} - \gamma_w$ = 10 kN/m^3	
	Pile diameter, d = 0.6 m	
	c_u = 96.19 kPa; α = 0.46	
	As = 3.14 × 0.6 × 6.5 = 12.25 m^2	
	Nc = 9 At = 0.28 m^2	
	Skin friction $Qs = \alpha\, c_u\, As$ = 0.46 × 96.19 × 12.25 = 542 kN	542
	p_t = 5 × 5.16 + 6 × 8.5 + 8 × 8.5 + 4 × 10 + 6.5 × 10 = 250 kN/m^2	
	End bearing, Qb = Nc pt At = 9 × 250 × 0.28 = 630 kN	630
	Sum =	2664
	FOS =	2.5
	Pile capacity (Axial) =	1066 kN

5.1.3.2.2 Lateral capacity

The lateral capacity should be determined by computer analysis using the pile as a stick model and soil subgrade modulus as spring support. The deflection should be limited from 7 to 12 mm to get allowable lateral load. However, actual lateral load capacity should be established by field load test.

In the absence of computer analysis, the following method can be used to get the lateral load at the pile head. Refer to IS 2911 (Part 1/Sec 2): 2010 – ANNEX – C.

The long flexible pile is treated as a cantilever fixed at some depth below the ground level.

TABLE 5.17
Modulus of subgrade reaction for cohesive soil, k_1 in kN/m³

		Unconfined compression strength, q_u	Range of k_1
SL No	**Soil consistency**	**kN/m²**	**kN/m³ × 10³**
1	Very soft	< 25	0
2	Soft	25 – 50	4.5 – 9
3	Medium stiff	50 – 100	9 – 18
4	Stiff	100 – 200	18 – 36
5	Very stiff	200 – 400	36 – 72
6	Hard	> 400	> 72

For piles in preloaded clay,
Stiffness factor, $R = {}^{4}\sqrt{}\ (Ec\ I / KB)$
$Ec = 5000\ \sqrt{fck} = 25000\ MN/m^2$ for $fck = 25$ MPa
$I = (3.14 \times 0.6^4) / 64 = 0.0063585\ m^4$
$K= (k_1 / 1.5) \times (0.3 / B) = (9 / 1.5) \times (0.3 / 0.6) = 3\ MN/m^4$
where $k_1 = 9\ MN/m^3$ (from Figure 5.17 above)
$B = 0.6$ m (pile diameter)
$R = {}^{4}\sqrt{}[(25000 \times 0.0063585) / (3 \times 0.6)] = 3$
From depth of fixity chart [Figure 2 determination of depth of fixity (IS 2911 Part 1 / Sec 2: 2010)],
We get L_1 /R 0.05 and $L_f / R = 2$ fixed headed in clay
$L_1 = e = 0.15$ m.
$L_f = 2 \times 3 = 6$ m
Pile head deflection = Y $Y = Q (e + L_f)\ ^3/12\ EI$ for the fixed-headed pile
and $Y = Q (e + L_f)\ ^3/3\ EI$ for free-headed pile
In this case,
$Y = [Q (e + L_f)\ ^3/12\ EI] \text{ x } 103$
$= Q \times [(6 + 0.15)^3 / (12 \times 25000 \times 0.0063585)] \times 1000$
$= 0.122\ Q$
Y = pile head deflection in mm
Q = lateral load in kN
$E = 25000000\ kN/m^2$
$I = 0.0063585\ m^4$
Lf = 6 m
e = 0.15 m

TABLE 5.18
Y vs Q

Y	Q
Mm	kN
5	41
6	49
7	57
8	66
10	82
12	98

Y = 0.122Q ; let us accept maximum allowable deflection = 10 mm,
So, for Y = 10 mm Q = **82** kN

Moment at the pile head (Mf) for lateral load (Q):

Moment at top of the pile, Mf = m. Q (e + L_f) /2 for free-headed pile,
where m = moment reduction factor

M = 0.83 × 82 × 6.15 / 2 = 209 kNm m = 0.83

[refer to Figure 3 Determination of reduction factors for computation of maximum moment in fixed-headed pile (IS 2911 Part 1/Sec2)]

Pile capacity (lateral), Q **= 82** kN and Mf = **209** kNm

5.1.3.2.3 Axial tension or pull out capacity

As per Tomlinson, pull out capacity, T = 50% (*) Ultimate skin resistance/FOS + weight of the pile

T = 0.5 × (2664−630) / 2.5 + (3.14 × 0.6^2 / 4) × 25 × 28 = 605 kN
FOS = 2.5
Pile capacity (uplift) = **605** kN
(*) It is a conservative approach; 50% loss is generally advised for short pile;

5.1.3.3 Summary of pile capacity as per geotechnical report

Pile diameter = 600 mm Length of pile = 28 mm

Pile capacity:

Axial compression = 900.kN [for foundation settlement within 15 mm]
Lateral = 82 kN
Uplift = 450 kN

The axial capacities should be checked by doing settlement analysis of pile groups and applying group efficiency factors. The allowable design capacity of the pile and its founding level should satisfy the permissible settlement limit.

In sites where piles carry negative friction due to imposed filling layer at the top and subsequent consolidation of soft clay at the bottom layers, the allowable capacity will be further reduced by deducting the calculated value of the negative friction. The method of calculating negative friction is illustrated in section 18.8 in Bowels.

However, the actual capacity shall be determined after the pile load test in the field.

5.1.3.4 Settlement analysis of pile group

The following calculation shows how to calculate the settlement of pile groups. In this analysis, a group of nine piles has been considered.

TABLE 5.19
Soil parameters for settlement calculation

Layer	Description	Depth (m)	N60	Cc	e_0	γ_{sat} kN/m³	Remarks
1	Gray soft silty clay	5	5	0.262	1.062	15.16	
2	Gray medium-stiff sandy silt	6	16			18.5	
3	Gray medium-dense silt and fine sand	8	22			18.5	
4	Gray very stiff silty clay	4	25	0.15	0.9	20	
5	Gray stiff hard silty clay	17	32	0.20	0.9	20	Founding level
6	Gray stiff hard silty clay	15	32	0.20	0.9		

FGL = 0.0 M   EGL = −1.0 M   GWL = 0.0 M
Pile cut-off level = (−) 2.5 M
Pile diameter = 0.6 M   length of the pile, Lp = 28 M
Pile group = 9G (nine pile group)   number of piles = 9
Group efficiency = 0.7   pile capacity = 900 kN (Axial)
Total load, P = 9 × 0.7 × 900 = 5670 kN

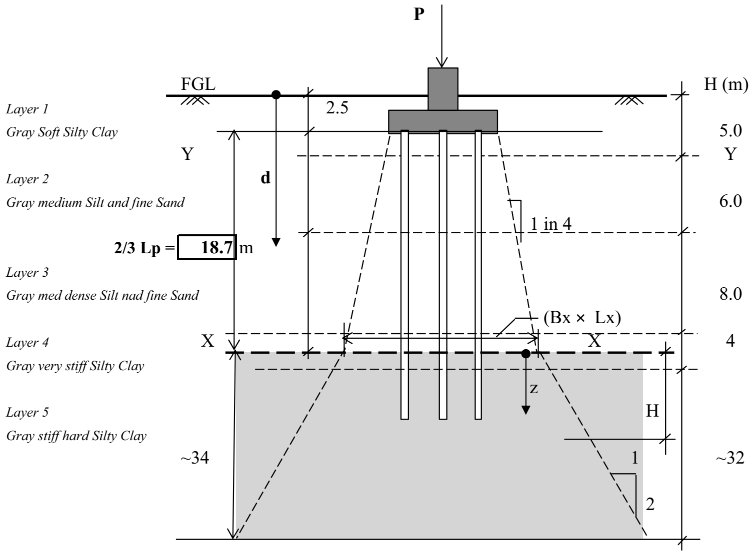

FIGURE 5.10 Pressure distribution below the pile group

Total settlement = $S_I + S_{II}$
Effective layer of settlement = H
[The depth at which the pressure increment is less than 10% of overburden pressure will provide the total thickness (H) of soil to be evaluated in the settlement computation. Refer to CL 9.8.2.3 in workshop manual Vol I]

Initial settlement, S_I (CL 9.8.2.1 womkshop Manual)

S_I = elastic compression of the pile = Qu. L / (A.Ec)
Ec = 25000 N/mm² A = (3.14 × 0.6²) / 4 = 0.28 m²
Qu = Qpa + αs. Qfa
where Qpa = axial load by bearing at the pile tip = 24 % × 900 = 216 kN
Qfa = axial load by shaft friction = 76 % × 900 = 684 kN
Qu = 216 + 0.5 × 684 = 558 kN Lp = 28 m
S_I = 558000 × 28000 / (282600 × 25000) = **2.2** mm

Consolidation settlement of cohesive layers below X-X, S_{II}:

$\Delta H = H\,[\{Cc / (1 + e_0)\} \times \log (p_0 + \Delta p) / p_0]$
H = Σ h = original thickness of the stratum in mm
Cc = compression index e_0 = initial void ratio
p_0 = effective overburden pressure at the midpoint of the compressible stratum prior to pressure increase in kPa
Δp = average change in pressure in the compressible stratum in kPa
ΔH= total settlement in mm
$p_0 = q + [\gamma sat.\ d - \gamma w.\ (GWL- d)]$
z = depth of point, where p_0 is taken, below X-X
Δp = P/A = area at the middle of the segment
q = surcharge above X-X = 30 kN/m² Bx = Lx = 4.35 + 0.5 × 18.67 = 13.7 m
Area at X-X = 13.7 × 13.7 = 188 m²

TABLE 5.20
Settlement in layers below X-X

Segment	h m	z m	$\gamma_{sat} - \gamma_w$ kN/m³	p_0 kN/m²	A m²	Δp kN/m²	Cc	e_0	ΔH mm	Total Deflection	$p_0/\Delta p$ [*]
1	1	0.5	10	175	202	28.07	0.15	0.9	5.101	10.80	**0.14**
2	1	1.5	10	185	231	24.54	0.20	0.9	5.694		**0.11**
3	1	2.5	10	195	262	21.60	0.2	0.9	4.804		0.08
4	1	3.5	10	205	296	19.17	0.2	0.9	4.086		0.06
5	1	4.5	10	215	331	17.12	0.2	0.9	3.502		0.05
6	1	5.5	10	225	369	15.38	0.2	0.9	3.023		0.04
7	1	6.5	10	235	408	13.90	0.2	0.9	2.626		0.03

[*] The depth at which the pressure increment is less than 10% of overburden pressure will provide the total thickness (H) of soil to be evaluated in the settlement computation.

Calculations breakup for Segment 1

$p_0 = 30 + (15.16-10) \times 5 + (18.5-10) \times 6 + (18.5 - 10) \times 8 = 175$ kN/m²

$A = (13.7 + 0.5) + (13.7 + 0.5) = 202$ m²

$\Delta p = 5670 / 202 = 28.07$ kN/m²

$\Delta H = 1\ [\{0.15 / (1 + 0.9)\} \times \log_{10} (175 + 28.07) / 175] \times 1000 = 5.101$ mm

Computation of consolidation settlement over 25 years:

From theory of consolidation, we get, $T = cv.\ t / H^2$

where t = time of consolidation = 25 years
H = effective thickness of the soil layer = 200 cm
Cv = coefficient of consolidation = 0.00035 cm² /sec
T = a dimensionless time factor

$T = cv.\ t / H^2 = 6.899$

Now, amount of consolidation, $U = \sqrt{(4\ T / p)}$ for $0 <= T <= 0.197$

Amount of consolidation, $U = 1 - (8 / p^2) \times e^{-p2.\ T/4}$ for $T > 0.197$ = 100%

So total settlement of the Pile group = 1.0 × 10.80 = 10.80 mm

Total settlement, $S_I + S_{II} = 2.2 + 10.80 = 13.01 < 15$ mm.

5.1.3.5 Structural design of pile

The following load combinations should be used for reinforcement and strength design:

1. Axial + lateral
2. Uplift + lateral

a) IS code method (IS: 456 and SP 16)

Let us consider the following design capacity for the pile:

Axial = 900 kN
Lateral = 82 kN

Uplift = 450 kN
M at top = 209 kNm
Pile diameter = 0.6 m

Concrete grade: M25 fck = 25 MPa
re-bars: Fe 500

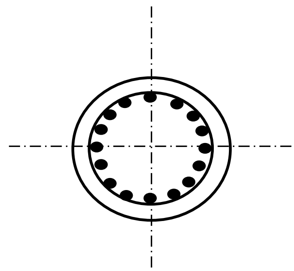

FIGURE 5.11 Pile reinforcement in the bore pile

Case I: Axial + lateral

P = 900 kN fy = 500 N/mm^2 fck = 25 kN/m^2
clear cover, d′ = 50 mm
D = 600 mm d′ / D = 0.083 M = 209 kNm
load factor = 1.7

Factored load

Pu = 900 × 1.7 = 1530 kN Pu/(fck.D^2) = 1530 × 10^3 / (25 × 600^2) = 0.17
Mu = 209 × 1.7 = 355 kNm Mu/(fck.D^3) = 355 × 10^6 / (25 × 600^3) = 0.07
From Chart 60 of SP 16 (refer to Chapter 5.13 Design Chart)
p /fck = 0.05 p = 0.05 × 25 = 1.25 %
As$_{\text{required}}$ = 0.0125 × 3.14 × 600^2 / 4 = 3534 mm^2
Minimum re-bars = 0.4 % of cross-sectional area = At$_{\text{min}}$ =1130 mm^2
Provide re-bar 16–20 diameter bars; As = 5027 mm^2 > 3534 mm^2 Hence, safe.

Case II: uplift + lateral*

*(*This combination may occur for limited cases, e.g., piles under bracing bay columns.)*

T = −450 kN fy = 500 MPa fck = 25 kPa d′ = 50 mm
D = 600 mm d′ / D = 0.1 M = 209 kNm

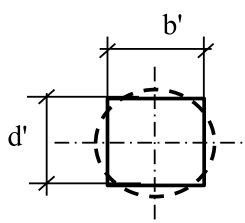

FIGURE 5.12 Equivalent square section

Dimension of a square section of equal area:

b′ = d′ = 532 mm

$Tu = -450 \times 1.7 = -765$ kN   Load factor = 1.7
$Mu = 209 \times 1.7 = 355$ kN
$Tu / (fck\ b'.\ d') = -765 \times 10^3 / (25 \times 532) = -0.11$
$Mu / (fck.b'.d'^2) = 355 \times 10^6 / (25 \times 532^2) = 0.09$
$p / fck = 0.08$ from Chart 83 SP 16 (refer to Chapter 5.13 Design Chart)
$p = 0.075 \times 25 = 1.875$ %
$As_{required} = 1.875 \times 532 \times 532 / 100 = 5307$ mm^2 At min = 1130 mm^2
Provide re-bar 8–20 diameter + 8–25 diameter bars; As = 6440 mm^2 > 5307 mm^2
Hence, safe.

The reinforcements calculated above in the above two cases are applicable for a length of 1.5 × Lf from top, i.e., 9 m long. The rest of the length will have minimum reinforcement – 12 nos. 12 diameter (As = 1357 mm^2 > 1130 mm^2).

Lateral ties/spiral bars

Provide lateral reinforcement in the form of spiral 8 mm diameter at 150 mm spacing

b) ACI Code method (ACI 318 and SP- 17–14 DA)

Pile sizing:
h = 0.6 m = 24 inch
γh = 0.5 m = 22 inch

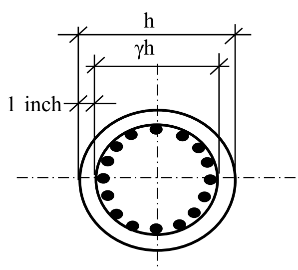

FIGURE 5.13 Pile reinforcement and cover

Materials:

$fc' = 25$ MPa = 4 ksi
fy = 60 ksi
Pu = 990 kN $\phi = 0.55$ $Pn = Pu/\phi = 1636$ kN
Mu = 209 kN $\phi = 0.65$ $Mnx = Mnx/\phi = 322$ kNm
ϕ = Strength reduction factor [ACI 318–19 (Table 21.2.1)]
Refer to SP 17–14 DA – ACI Handbook for the Reinforced Concrete Design Handbook
Column Interaction Diagram C4 – 60.9,

$Ag = 3.14 \times 24^2 / 4 = 452$ in^2
$Kn = Pn / fc'.Ag = 1636 / (4 \times 452) = 0.9$
$Rn = Pn\ e / fc'\ Ag\ h = 322 / (4 \times 452 \times 24) = 0.01$
$\gamma = 22 / 24 = 0.917$
$\rho = 0.01$ from the interaction diagram mentioned above,
$Ast = 0.01 \times 452 = 4.52$ in^2
Provide 16 nos. 20 diameter   Ast = 5027 mm^2 = 7.791 in^2 > 4.52 in^2
Hence, safe.
Provide lateral reinforcement in the form of spiral 8 mm diameter at 150 mm spacing.

5.2 DESIGN OF PILE CAPS

Pile caps are thick foundation slabs resting on a group of piles. Reinforced concrete building columns or pedestals bearing structural steel column bases are cast on top of the pile cap. Reinforcement bars of columns are embedded into the pile cap (or embedded dowel bars projected upward) to ensure a monolithic joint while transferring forces from columns or pedestals to the group of piles. Hence the pile caps should have adequate thickness and be designed with necessary reinforcement steel bars to carry the moment and shear similar to a footing slab resting on the ground. The strength design should be done as a footing or wide beam spanning over piles.

5.2.1 Pile cap for 3 pile group

Design parameters:

Finish-grade level, FGL = 0.3 M
Top of pedestal, TOP = 1.2 M
Top of pile cap, TOC = − 1.6 M
Bottom of pile cap, BOC = − 2.5 M
Groundwater level (*), GWL = − 3 M
Unit weight of concrete γconc = 25 kN/m^3
Unit weight of soil, γsoil = 16 kN/m^3
Concrete cover = 75 mm
Max bar diameter, ϕ diameter = 16 mm
Pile diameter = 400 mm

Pile capacity:
Axial compression = 400 kN
Tension = 200 kN
Lateral = 22 kN

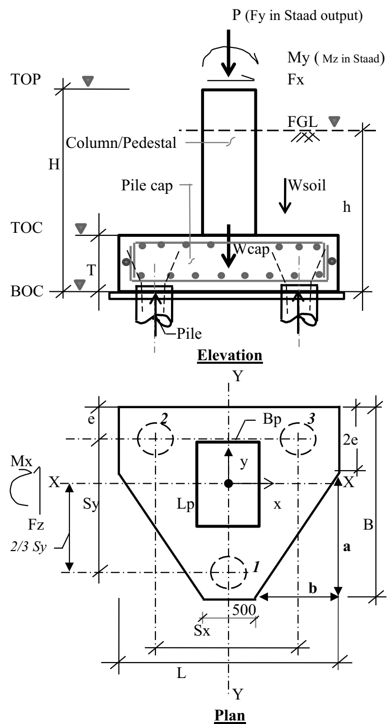

FIGURE 5.14 Pile cap foundation

Pile cap and pedestal dimensions

Lp = 0.9 M
Bp = 0.6 M
H = 1.2 + 2.5 = 3.7 M
h = 0.3 + 2.5 = 2.8 M
L = 1.2 + 2 × 0.35 = 1.9 M
B = 1.7 + 2 × 0.35 = 2.4 M
T = 0.9 M
e′ = 0.35 M
Effective thickness, d = 0.9−0.075−0.5 × 0.016 = 0.817 M
Sx = 1.2 M (3 × pile diameter)   Sy = 1.7 M   a = 1.7 M   b = 0.7 M

Grade of concrete = M25   fck = 25 Mpa (cube strength)
Reinforcement steel = Fe415   fy = 415 MPa

(Note: If GWL is above the bottom of the pile cap, then submerged weight of soil and concrete to be considered.)*

Wcap = [(1.9 × 2.4) − (1.7 × 0.7)] × 0.9 × 25 + 0.9 × 0.6 × (3.7−0.9) × 25 = 114 kN
Wsoil = [1.9 × 2.4−1.7 × 0.7−0.9 × 0.6] × (2.8−0.9) × 16 = 86 kN

Pile group property:

No. of pile (n) = 3
$x / \Sigma x^2 = (0.5 \times 1.2) / [2 \times (0.5 \times 1.2)^2] = 0.833\ m^3$
$y / \Sigma y^2 = 0.67 \times 1.7 / [2 \times (0.33 \times 1.7)^2 + 1 \times (0.67 \times 1.7)^2] = 0.592\ m^3$

5.2.1.1 Column/pedestal load

TABLE 5.21
Design load for load combinations Case 101

	Horizontal	Vertical	Horizontal	Moment			
Load case (#)	**Fx (kN)**	**Fy (kN)**	**Fy (kN)**	**Mx kNm**	**My kNm**	**Mz kNm**	**Load combination case**
101	0.001	779	1.7	2.9	0	−0.001	101 combination – 1 dead + 1 live
Col. 1	*2*	*3*	*4*	*5*	*6*	*7*	*8*

Note: # reference cell

TABLE 5.22
Load on the pile group

	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Item	0	780	1.71	2.885	0	0
Wcap		114				
Wsoil		86				
Fz.H				6.35		
Fx.H					0.0037	
Sum	**0**	**980**	**1.7**	**9**	**0**	**0**
		P		**Mx**		**My**

Note: for groundwater below the pile cap.

Maximum reaction in the pile

Axial = $P / n + Mx.y / \Sigma y^2 + Mz.x / \Sigma x^2$ = 327 + 5.46 + 0 = 332 kN;	Safe in compression
Tension = $P / n - Mx.y / \Sigma y^2 - Mz.x / \Sigma x^2$ = 327−5.46 − 0 = 321 kN;	No tension/uplift
Lateral = $\sqrt{(Fx^2 + Fz^2)} / n$ = 1.7 / 3 = 0.57 kN;	Safe in shear.

The reader should note that the above calculation has been done for a column load combination case, say load case 101 only.
The designer can see results of multiple load combination inputs in a tabular form, by using an MS Excel (Microsoft)-Data-What-if analysis-Data table.

Following steps are suggested:

I. *Prepare a summary of the load table (see* Table 5.23*) showing load case number and values for all load combination cases from the building frame analysis result – support reactions.*
II. *In Table 5.21, write the load case number in the "Reference Cell" – Column 1.*
III. *Now design load data in Columns 2 to 7 should be filled in from* Table 5.23.

Using VLOOKUP comment, please fill up load values in Columns 2 to 7 corresponding to the load case number given in the reference cell (Column 1).
For example,

Look up value: The reference cell as marked in Table 5.21 *(Column 1)*
Table array: Table 5.23 – all cells
Column Index number: 3 to 8, whichever may be applicable.
Range lookup: false.

IV. *Now get pile reactions for all the load combination cases in* Table 5.23*, using the Data Table option in Excel (Data > What-if > Data Table). The results are presented in* Table 5.24.

Here is the workout example for all load cases.

TABLE 5.23
Column base load for all load combinations

1	2	3	4	5	6	7	8
		Horizontal	Vertical	Horizontal	Moment		
Load case no	Load case no. and description	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
101	101 combination – 1 dead + 1 live	0.0	779.7	1.7	2.9	0.0	0.0
102	102 combination – 1 dead + 0.75 live	0.0	716.2	1.6	2.7	0.0	0.0
103	103 combination – 1 dead + 1 wind (1)	−13.9	515.2	−3.5	−10.6	0.0	60.2
104	104 combination – 1 dead + 1 wind (2)	13.9	529.4	−3.7	−11.0	0.0	−60.2
105	105 combination – 1 dead + 1 wind (3)	9.9	520.0	−2.7	−8.7	0.0	−8.7
106	106 combination – 1 dead + 1 wind (4)	9.9	522.7	−3.8	−11.6	0.0	−8.7

TABLE 5.23 (*Continued*)
Column base load for all load combinations

1	2	3	4	5	6	7	8
		Horizontal	Vertical	Horizontal	Moment		
Load case no	Load case no. and description	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
107	107 combination – 1 dead + 1 wind (5)	–19.5	516.7	–1.4	–4.9	0.0	64.9
108	108 combination – 1 dead + 1 wind (6)	2.3	525.7	–1.4	–5.0	0.0	–9.0
109	109 combination – 1 dead + 1 wind (7)	4.2	522.3	–2.2	–7.4	0.0	–3.6
110	110 combination – 1 dead + 1 wind (8)	4.2	527.0	–6.6	–18.9	0.0	–3.6
111	111 combination – 1 dead + 0.7 seismic-H (1)	–30.7	474.7	1.8	3.5	0.0	321.8
112	112 combination – 1 dead + 0.7 seismic-H (2)	0.0	530.9	–8.5	–24.4	0.0	0.0
113	113 combination – 1 dead + –0.7 seismic-H (1)	30.7	576.7	0.8	0.9	0.0	–321.8
114	114 combination – 1 dead + –0.7 seismic-H (2)	0.0	520.5	11.0	28.8	0.0	0.0
115	115 combination – 1 dead + 0.75 live + 0.75 wind (1)	–10.4	708.4	–2.0	–6.9	0.0	45.1
116	116 combination – 1 dead + 0.75 live + 0.75 wind (2)	10.4	719.0	–2.1	–7.2	0.0	–45.1
117	117 combination – 1 dead + 0.75 live + 0.75 wind (3)	7.4	712.0	–1.4	–5.5	0.0	–6.5
118	118 combination – 1 dead + 0.75 live + 0.75 wind (4)	7.4	714.0	–2.2	–7.6	0.0	–6.5
119	119 combination – 1 dead + 0.75 live + 0.75 wind (5)	–14.7	709.5	–0.4	–2.6	0.0	48.7
120	120 combination – 1 dead + 0.75 live + 0.75 wind (6)	1.7	716.2	–0.4	–2.7	0.0	–6.8
121	121 combination – 1 dead + 0.75 live + 0.75 wind (7)	3.2	713.7	–1.0	–4.5	0.0	–2.7
122	122 combination – 1 dead + 0.75 live + 0.75 wind (8)	3.2	717.2	–4.3	–13.1	0.0	–2.7
123	123 combination – 1 dead + 0.75 live + 0.525 seismic-H (1)	–23.0	678.0	2.0	3.7	0.0	241.3
124	124 combination – 1 dead + 0.75 live + 0.525 seismic-H (2)	0.0	720.2	–5.7	–17.2	0.0	0.0
125	125 combination – 1 dead + 0.75 live + –0.525 seismic-H (1)	23.0	754.5	1.2	1.8	0.0	–241.3

(*Continued*)

TABLE 5.23 (*Continued*)
Column base load for all load combinations

1	2	3	4	5	6	7	8
		Horizontal	Vertical	Horizontal	Moment		
Load case no	Load case no. and description	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
126	126 combination – 1 dead + 0.75 live + –0.525 seismic-H (2)	0.0	712.3	8.9	22.6	0.0	0.0
127	127 combination – 0.6 dead + 1 wind (1)	–13.9	305.0	–4.0	–11.5	0.0	60.2
128	128 combination – 0.6 dead + 1 wind (2)	13.9	319.1	–4.2	–11.9	0.0	–60.2
129	129 combination – 0.6 dead + 1 wind (3)	9.9	309.7	–3.3	–9.6	0.0	–8.7
130	130 combination – 0.6 dead + 1 wind (4)	9.9	312.4	–4.3	–12.5	0.0	–8.7
131	131 combination – 0.6 dead + 1 wind (5)	–19.5	306.4	–1.9	–5.8	0.0	64.9
132	132 combination – 0.6 dead + 1 wind (6)	2.3	315.4	–1.9	–5.9	0.0	–9.0
133	133 combination – 0.6 dead + 1 wind (7)	4.2	312.0	–2.8	–8.3	0.0	–3.6
134	134 combination – 0.6 dead + 1 wind (8)	4.2	316.7	–7.1	–19.8	0.0	–3.6
135	135 combination – 0.6 dead + 0.7 seismic-H (1)	–30.7	264.4	1.3	2.6	0.0	321.8
136	136 combination – 0.6 dead + 0.7 seismic-H (2)	0.0	320.7	–9.0	–25.3	0.0	0.0
137	137 combination – 0.6 dead + –0.7 seismic-H (1)	30.7	366.4	0.3	0.1	0.0	–321.8
138	138 combination – 0.6 dead + –0.7 seismic-H (2)	0.0	310.2	10.5	27.9	0.0	0.0

5.2.1.2 Strength design of the pile cap

The pile cap is acting as a wide beam. The maximum bending moment will be at the face of the column or pedestal. It is assumed that the reaction from any pile is concentrated at the center of pile. The critical section for shear will be at a distance equal to half the effective depth of the pile cap slab from the face of the pedestal. A reduction factor (α) may be applied to determine shear on the critical section as stated below.

Pile load reduction factor, α: (IS 456: 2000 34.2.4.2)
dp = pile diameter.

TABLE 5.24
Pile reactions for all load cases.

Load case	Observation			Axial kN	Tension kN	Shear kN
101	Safe in compression	No tension	Safe in shear	**332**	**321**	**0.6**
102	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
103	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
104	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
105	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
106	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
107	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
108	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
109	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
110	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
111	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
112	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
113	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
114	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
115	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
116	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
117	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
118	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
119	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
120	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
121	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
122	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
123	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
124	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
125	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
126	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
127	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
128	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
129	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
130	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
131	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
132	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
133	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
134	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
135	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
136	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
137	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
138	Safe in compression	No tension	Safe in shear	311	299.97	0.5363
			max	**332**	**320.78**	**0.5717**

Let us design the pile cap for load case 101, dead load + live load, which gives the maximum pile reaction of all the load cases.

$\alpha = 1$ (full shear when the pile center is dp/2 or more than dp/2 outside the critical section).
$\alpha = 0$ (no shear, when the pile center is located at dp/2 or more inside the critical section).
$\alpha = 0$ to 1 (for intermediate positions of the pile center, the portion of the pile reaction to be assumed as producing shear on the section shall be based on straight line interpolation between full value at dp/2 outside the section and zero value at dp/2 inside the section.)

From Figure 5.34 above, let us compute the maximum design moment and shear for the strength design of the pile cap slab along both axes and provide reinforcement steel bars accordingly.

a) **Along the Y axis:**
Y = pile center distance from the face of the pedestal
X = critical shear plane from the face of the pedestal
d = effective depth of the pile cap

Reaction from piles 2 and 3 – 2 piles:

Lp/2 = 0.45 m Sy/3 = 0.567 m pile center is outside the face of the pedestal.
X = d/2 = 0.409 m
Y = Sy / 3 ~ 0.5 Lp = 0.117m dp/2 = 0.4 / 2 = 0.2 m
X − Y = 0.409 − 0.117 = 0.29 m > dp / 2.
Pile center in more than dp/2 inside the critical section, these piles will not produce any shear.

Reaction from pile 1

Pile center distance: X = d /2 = 0.409 m
Y = 2/3 Sy − 0.5 Lp = 0.67 × 1.7–0.5 × 0.9 = 0.69 m
Y − X = 0.69–0.409 = 0.28 m
dp / 2 = 0.4 / 2 = 0.2 m
As the pile center in more than dp/2 outside the critical section, these piles will produce full shear.
Effective No. of piles outside the critical shear plane = 1
Total shear = 1 × 332 = 332 kN
Effective width of pile cap at face of pedestal = L' = 1.22 m
Self-weight of the pile cap projecting outside the face of the pedestal
= [0.5 × (0.5 + 1.22) × 0.69] × 0.9 × 25 = 33 kN.
CG from the face of the pedestal = (0.69 / 3) × (1.22 + 2 × 0.5) / (1.22 + 0.5) = 0.29 m
CG = h/3 [(b + 2a) / (b + a)]
h = 0.69 m a = 0.5 m b = 1.22 m
Moment = 332 × 0.28–33 × 0.29 = 83 kNm (moment reduced by self-weight of the cap)
i.e., 83 / 1.22 = 68 kNm per meter wide.
Shear, V = (332 − 33) / 1.22 = 245 kN/meter wide.

b) Along the X axis:

Y = pile center distance from the face of the pedestal
X= critical shear plane from the face of the pedestal
d = effective depth

Reaction from pile no 2 or 3 (1 pile)

Y= 0.5 Sx − 0.5 Bp = 0.5 × 1.2−0.5 × 0.6 = 0.3 m
X = d/2 = 0.817 / 2 = 0.409 m
e_1 = X − Y = 0.409−0.3 = 0.109 m
dp / 2 = 0.4 / 2 = 0.2 m

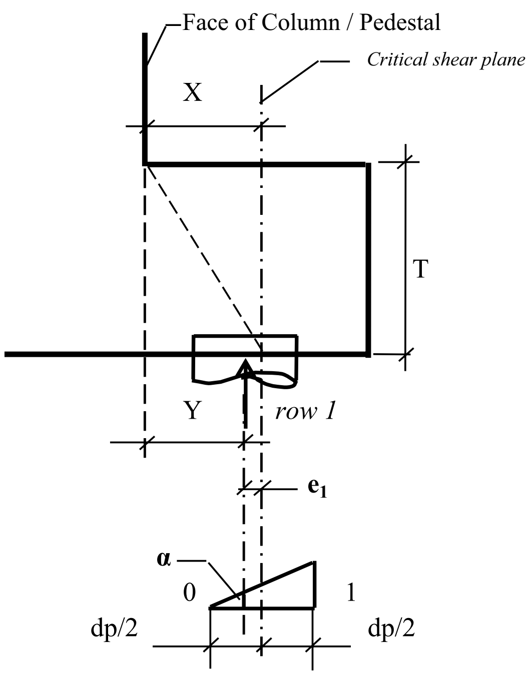

FIGURE 5.15 Part section along the Y axis

The pile center is inside the critical shear plane but within dp/2, so the pile reaction will be reduced.
Reduction factor, α = (0.2 − 0.109) × (1 / 0.4) = 0.23
Effective No. of piles = 1
Total shear = 1 × 0.23 × 332 = 76 kN
Effective width for resisting pile reaction at support = 2 × 0.35 + 0.5 × 1.7 = 1.55 m
Self-weight of the pile cap projecting outside the face of the pedestal = A + B = 23.63 kN
A = (0.35 × 2) × 0.5 × (1.9−0.6) × 0.9 × 25 = 10.24 kN

$B = 0.5 \times (0.7 \times 1.7) \times 0.9 \times 25 = 13.39$ kN
CG distance from the face of the pedestal,
$A = [0.5 \times (1.9 - 0.6)] /2 = 0.325$ m
$B = 0.7 / 3 = 0.23$ m
Moment $= 76 \times 0.3 - 10.24 \times 0.325 - 13.39 \times 0.23 = 16.39$ kNm
i.e., $16.39 / 1.55 = 11$ kNm/meter
Shear $= (76-10.24-13.39) / 1.55 = 34$ kN/meter

Provide the following reinforcement bars in the pile cap:

Along the Y axis long bars – 16 diameter @ 150 mm c/c (Ast = 1341 mm^2 /m)
Along the X axis transverse bars – 16 diameter @ 150 mm c/c (Ast = 1341 mm^2 /m)

Strength design as per IS code – IS: 456 and SP 16

Long bars: along the Y axis load factor = 1.5
Moment at face of columns = 68 kNm /m (Mx)
d provided = 817 mm
$Mu /bd^2 = 1.5 \times 68 \times 1000000 / (1000 \times 817^2) = 0.15$
pt = 0.084 % for M25 (SP 16 Table 3)
$Ast_{reqd} = (0.084/100) \times 1000 \times 817 = 686\ mm^2/m$ width
$Ast_{provided} = 1341\ mm^2/m$ width $> Ast_{reqd}$; Safe.
Ast minimum $= (0.12 / 100) \times 1000 \times 817 = 980\ mm^2/m$ (IS 456: 2000 26.5.2)

Shear:

$pt_{provided} = 1341 \times 100 / (1000 \times 817) = 0.16$ %
$\tau c = 0.30$ for pt = 0.16% (IS 456:2000 Table 19)
Shear capacity of concrete $= \tau c$ b d $= 0.297 \times 1000 \times 817 / 1000 = 243$ kN
Design shear, V = 245 kN
$Vu = 1.5 \times 245 = 368$ kN $> \tau c$ b d provide shear reinforcement.

Let us provide shear reinforcement in the form of closed stirrups at equal spacing along the Y direction – 2 Legged 12 mm diameter @ 150 centers

Vus = shear capacity of vertical stirrups = 0.87 fy Asv d / sv (IS 456:2000 – 40.4 a)
Asv = cross-sectional area of stirrup $= 2 \times 3.14 \times 12^2 / 4 = 226\ mm^2$
sv = spacing of stirrups = 150 mm c/c.
d = effective depth of the pile cap = 817 mm
fy = 415 MPa
$Vus = 0.87 \times 415 \times 226 \times 817 / 150 = 444$ kN
Shear capacity = Vus + τc.b.d = 444 + 243 = 687 kN > Vu; safe.

[Shear reinforcement may be provided as closed link around the vertical sides or single link bars within the critical zone of shear].

Transverse bars: along the X axis

Moment at the face of column = 11 kNm/m (My)
d_{reqd} = 96 mm d_{prov} = 817 mm
Mu /bd^2 = 0.02 pt = 0.084 (M25)
$Ast_{reqd.}$ = 686 mm^2/m width
$Ast_{provided}$ = 1341 mm^2/m width
Ast_{min} = 980 mm^2/m width

Shear

$pt_{provided}$ = 1341 × 100 / (1000 × 817) = 0.16 %
τc = 0.30
Shear capacity = τc.b.d = 0.297 × 1000 × 817 / 1000 = 243 kN
V = 34 kN
Vu = 1.5 × 34 = 51 kN < τc.b.d; Ok. Shear reinforcement not required.

Shrinkage reinforcement at the top layer:

Provide 12 diameter @ 150 mm c/c. both ways (Ast = 754 mm^2 /m) > 0.5 Minm. Reinforcement (0.5 × 980 = 490 mm^2 /m.)

[Note: Side face reinforcement should be provided for pile caps of depth more than 600 mm.]

Design of the pedestal should be the same as the design of the column (Chapter 5.7).

5.2.1.3 Strength design in the ACI Code method

Reference code: ACI 318–19 **Unit: SI metric; stress in Mpa**

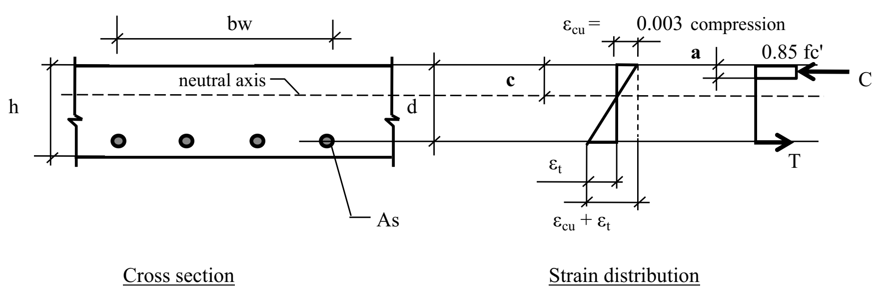

FIGURE 5.16 Stress and strain diagram for the foundation slab

Design parameters:

One-way slab (cantilever type)

fc′ =20 MPa	bw = 1000 mm
fy = 415 MPa	h = 900 mm (> span/10)
Es = 200000 MPa	d′ = 75 mm cover
Bar diameter = 16 mm	d = effective depth = 817 mm

Design load	*Factored load (*)*
Mx = 68 kNm	Mu long = 102 kNm
My = 11 kNm	Mu Tranv = 17 kNm
V = 245 kN	Vu = 368 kN

(*) Factored load should be determined according to ACI 318 – Table 5.3.1

Provide main bars:
Along the Y axis – 16 diameter @ 150 mm c/c (Ast = 1340 mm^2 /m width)
Along the X axis – 16 diameter @ 150 mm c/c (Ast = 1340 mm^2 /m width)
Minimum reinforcement: ACI 318–19 (7.6.1.1)
In flexure – As min = 0.0018 Ag = 0.0018 × 1000 × 900 = 1620 mm^2

To find out NA depth

Maximum usable strain at concrete compression fiber, $\varepsilon_{cu} = 0.003$
Net tensile strain at steel reinforcement,
$\varepsilon_t = 0.005$ [>= εt y + 0.003] ACI 318–19 (21.2.2.1)
ε_{ty} = fy / Es = 0.00208 Tension reinforcement yielded.
Depth of the neutral axis from the top, $c = \varepsilon_{cu}.\ d / (\varepsilon_{cu} + \varepsilon_t)$
= 0.003 × 817 / (0.003 + 0.005) = 306 mm
a = β × c ACI 318–19 (21.2.2.4.1)
β = 0.85 ACI 318–19 (Table 22.2.2.4.3)
a = 0.85 × 306 = 260 mm

Long bars along the Y axis

Flexural reinforcement as a singly reinforced slab
Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)
Mu = 102 kNm tension-controlled
Required nominal strength, Mn = Mu/ ϕ = 113 kNm ϕ Mn > Mu
From the stress block above, a = 260 mm c = 306 mm $\varepsilon_{cu} = 0.003$
C = 0.85 fc′. a. bw = 0.85 × 20 × 260 × 1000 / 1000 = 4420 kN

Mn_1 = C (d – a/2) = 4420 × (817 – 0.5 × 260) / 1000 = 3037 kNm > Mn; singly reinforced.
$As_{required}$ = Mn /fy. (d – a/2) = 113 × 1000000 / 415 × (817 – 0.5 × 260) = 398 mm^2
Reinforcement provided = 1340 mm^2 > $As_{required}$; Safe.

Transverse bar along the X axis

Flexural reinforcement as a singly reinforced slab
Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)
Mu = 17 kNm tension-controlled
Required nominal strength, Mn = Mu/ ϕ = 18 kNm ϕ Mn > Mu
C = 0.85 fc′. a. bw = 0.85 × 20 × 260 × 1000 / 1000 = 4420 kN

Mn_1 = C (d – a/2) = 4420 × (817 – 0.5 × 260) / 1000 = 3037 kNm > Mn; singly reinforced.
$As_{required}$ = Mn /fy. (d – a/2) = 18 × 1000000 / 415 × (817 – 0.5 × 260) = 64 mm^2
Reinforcement provided = 1340 mm^2 > $As_{required}$; Safe.

Shear reinforcement

Strength reduction factor, $\phi = 0.75$ ACI 318–19 (Table 21.2.1)
$Vu = 368$ kN
Required nominal strength, $Vn = Vu / \phi = 490$ kN $\phi Vn >= Vu$
Shear capacity = Vc ACI 318–19 (Table 22.5.5.1a)

$Vc = 0.17\ \lambda \sqrt{fc'}.\ bw.\ d = 0.17 \times 0.68 \times \sqrt{20} \times 1000 \times 817 / 1000 = 425$ kN.
$\lambda = \sqrt{2 / (1 + 0.004\ d)} = 0.68 <= 1$ ACI 318–19 (22.5.5.1.3)

Here, Vc < Vn Provide Shear reinforcement.

$Vs = Av\ f_{yt}\ d / s$ CI 318–19 (22.5.8.5.3)
s = longitudinal spacing
Av = effective area of bars
Let us provide vertical stirrups two-legged – 16 mm diameter @ 200 mm centers (the bars may be provided in single links at equal spacing on both sides within shear zone)
Vs= shear capacity of vertical stirrups
Av = effective area of all stirrup bar legs = $2 \times 3.14 \times 16^2 /4 = 402$ mm^2

s = longitudinal spacing of shear reinforcement = 200 mm c/c
d = effective depth of the pile cap slab = 817 mm
fyt = fy = 415 MPa
$Vs = Av\ fyt\ d / s = (402 \times 415 \times 817 / 200) / 1000 = 682$ kN ACI 318–19 (22.5.8.5.3)
Shear capacity, Vc +Vs = 425 + 682 = 1107 > Vn; Safe. ACI 318–19 (22.5.8.1)
Maximum allowable shear in this C/S of the slab, Vu_{max} ACI 318–19 (22.5.1.2)
$Vu_{max} = \phi\ (Vc + 0.66 \sqrt{fc}.\ bw.d) = 1809$ kN > Vu; okay.

Shrinkage reinforcement bars at the top layer ACI 318–19 (24.4.3.2)

Shrinkage reinforcement (0.0018Ag) should be distributed in two layers. There are tension reinforcement bars in the bottom layer, so provide half of minimum re-bars (0.0009Ag) at the top.

Provide 16 diameter @ 200 c/c (Ast = 1006 mm^2 /m) > 0.0009Ag = 810 mm^2 /m).
Design of the pedestal shall be the same as defined in ACI – 318.

5.2.1.4 Anchor bolt

The dimensional details of anchor bolts will be available in the steel base plate drawing for the pile cap bearing steel column. However, the designer should check the concrete breakout strength or rupture of the pedestal over the embedded length of the anchor bolt.

The method of concrete breakout strength determination is available in ACI 318R -14 – Building Code Requirements for Structural Concrete and Commentary.

5.2.2 Pile cap for 4 pile group

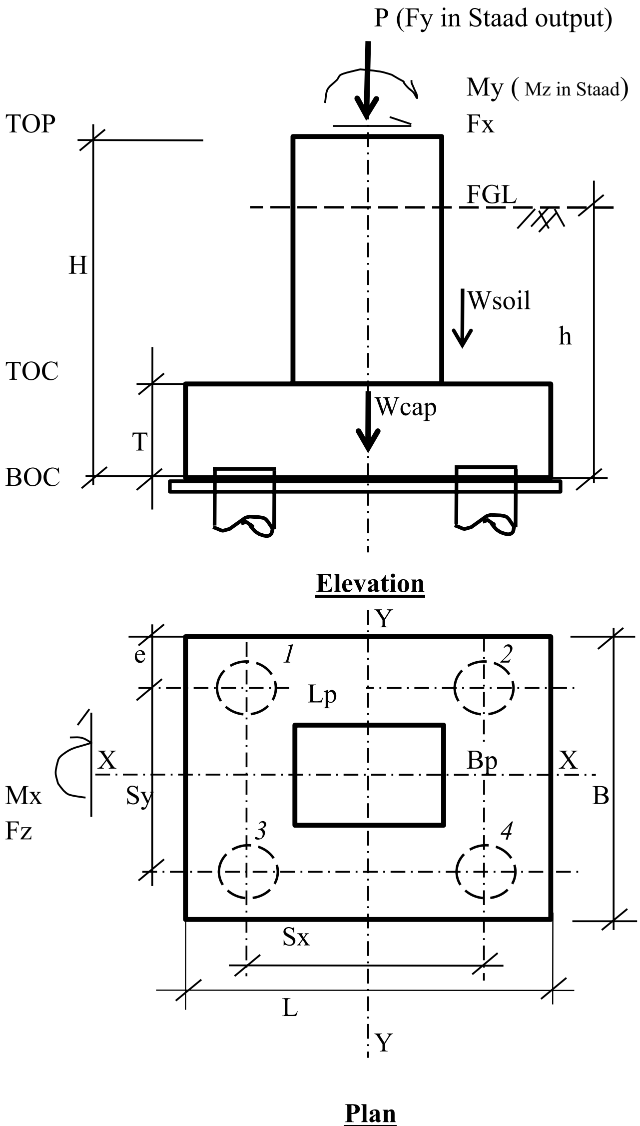

FIGURE 5.17 Pile cap foundation – 4 pile group

Design parameters

Levels:

FGL = 0.3 M TOP = 1.2 M TOC = −1.4 BOC = −2.2 M
GWL = −3.0 M

Pile cap dimensions:

Sx = 1.2 m	Sy = 1.2 m	L = 1.9 m	B = 1.9 m
T = 0.8 m	e = 0.35 m	h = 2.5 m	H = 3.4 m
Lp = 0.9 m	Bp = 0.6 m	d, effective depth = 0.717 m.	
Wcap = 107 kN	Wsoil = 84 kN		
Cover = 75 mm	Maximum bar diameter, ϕ diameter = 16 mm		

Materials:

γconc = 25 kN/m^3 γsoil = 16 kN/m^3 Grade of Concrete = M25
fck = 25 MPa (cube strength) Reinforcement steel = Fe415 fy = 415 MPa

Pile capacity:

Pile diameter = 400 mm
Axial = 400 kN (comp) Tension = 200 kN Lateral = 22 kN

5.2.2.1 Column/pedestal load

TABLE 5.25
Design load

	Horizontal	Vertical	Horizontal	Moment			
Load Case	**Fx (kN)**	**Fy (kN)**	**Fy (kN)**	**Mx kNm**	**My kNm**	**Mz kNm**	**Load combination case**
101	0	1370	0.173	0.383	0	0	101 Comb – 1 Dead + 1 Live

TABLE 5.26
Load on pile group:

	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Item	**0**	**1370**	**0.173**			
Wcap		107				
Wsoil		84				
Fz.H				0.59		
Fx.H					0	
Sum	**0**	**1561**	**0.2**	**0.59**	**0**	**0**
		P		**Mx**		**My**

Note
1. Column loads are taken from a building analysis report (STADD output)]
2. If GWL is above bottom of pile cap, then submerged weight of soil and concrete to be considered.

Pile group property

Nos. of pile (n) = **4**

$x / \Sigma x^2 = (0.5 \times 1.2) / [4 \times (0.5 \text{ x } 1.2)^2] = 0.42 \text{ m}^3$
$y / \Sigma y^2 = (0.5 \times 1.2) / [4 \times (0.5 \text{ x } 1.2)^2] = 0.42 \text{ m}^3$

Max load per pile

Axial = $P/n + Mx.y/\Sigma y^2 + Mz.x/\Sigma x^2 = 390 + 0.25 + 0 = 391$ kN < 400 kN; safe
Tension = $P/n - Mx.y/\Sigma y^2 - Mz.x/\Sigma x^2 = 390-0.25-0 = 390$ kN; No uplift.
Lateral = $\sqrt{(Fx^2 + Fz^2)} / n = 0.173 / 4 = 0.043$ kN < 22 kN; Safe.

Strength design of pile cap

a) Along the Y axis:

Y = pile center distance from the face of the pedestal.
X = critical shear plane from the face of the pedestal
d = effective depth of the pile cap

Reaction from piles 1 and 2 – (2 piles)

X = d / 2 = 0.359 m Y = (1.2–0.6) / 2 = 0.30 m

Pile center is inside the critical shear plane.
$e_1 = X - Y = 0.359 - 0.3 = 0.059$ m < dp / 2
dp / 2 = 0.4 /2 = 0.2 m
As the pile center is inside the critical shear plane but less than dp/2 away, so the shear will be proportionally reduced.

Reduction factor = α
$\alpha = (0.2 - 0.059) \times (1 / 0.4) = 0.35$
Effective No. of piles = 2
Total shear = $2 \times 0.35 \times 391 = 273$ kN

Weight of the pile cap projecting outside pedestal:

Width = 1.9 m
Projection = (1.9–0.6) / 2 = 0.65 m
Self-weight of pile cap projecting outside face of pedestal =$1.9 \times 0.65 \times 0.8 \times 25 = 25$ kN

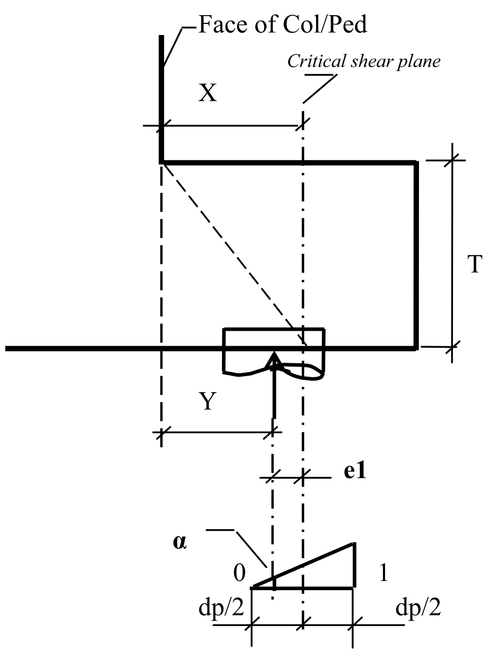

FIGURE 5.18 Part section along Y-Y

Moment = 273 × 0.3 − 25 × 0.5 × 0.65 = 74 kNm i.e., 74 / 1.9 = 39 kNm meter wide
Shear = 273−25 = 248 kN, i.e., 248 / 1.9 = 131 kN/meter width.

b) Along the X axis:

Y = pile center distance from the face of the pedestal
X= critical shear plane from the face of the pedestal
d = effective depth

Reaction from pile 2 and 3

Y= 0.5 Sx − S0.5 Lp = 0.5 × 1.2−0.5 × 0.9 = 0.15 m
X = d / 2 = 0.717 / 2 = 0.209 m
Pile center in inside the critical shear plane.
e_1 = X − Y = 0.359 − 0.15 = 0.209 m dp / 2 = 0.4 / 2 = 0.2 m
Pile center is the inside critical shear plane and more than dp/2 away; hence will not produce any shear.
Effective No of piles = 2
Distance of pile center outside the pedestal face = (1.2−0.9) / 2 = 0.15 m
Weight of the pile cap projecting the outside pedestal:
Width = 1.9 m projection = (1.9−0.9) / 2 = 0.5 m
Self-weight of pile cap projecting outside face of pedestal

19 × 0.5 × 0.8 × 25 = 19 kN

Projection outside the shear plane = 0.5−0.359 = 0.141 m

Moment = 2 × 390.5 × 0.15 − 19 × 0.5× 0.5 = 112 kNm
i.e., 112 / 1.9 = 59 kNm/meter wide
Shear = 0 kN/m width
Provide following reinforcement bars in the bottom layer of the pile cap:

Along the Y axis- long bars	16 diameter @ 225 mm c/c (Ast = 894 mm^2 /m)
Along the X axis - transverse bars	16 diameter @ 225 mm c/c (Ast = 894 mm^2 /m)

As Per IS Code – IS: 456 and SP 16

a) Along the Y axis Load factor = 1.5
Moment at face of column = 39 kNm /m
d provided = 717 mm
Mu /bd^2 = 1.5 × 39× 1000000 / (1000 × 717^2) = 0.11
pt = 0.084 % for M25 (SP 16 Table 3)

Ast $_{reqd}$ = (0.084/100) × 1000 × 717 = 602 mm^2/m width
Ast $_{provided}$ = 894 mm^2/m width > Ast $_{reqd}$; Safe.
Ast minimum = (0.12 / 100) × 1000 × 717 = 860 mm^2/m (IS 456: 2000 26.5.2)

Shear:

$pt_{provided}$ = 894 × 100 / (1000 × 717) = 0.12 %
τc = 0.29 for pt = 0.12% (IS 456:2000 Table 19)
Shear capacity of concrete = τc b d = 0.29 × 1000 × 717 / 1000 = 208 kN
Design shear, V = 131 kN
Vu = 1.5 × 131 = 197 kN < τc b d Shear reinforcement is not required.

b) Along the X axis
Moment at face of column = 59 kNm/m
d_{prov} = 717 mm
Mu /bd^2 = 0.17 pt = 0.084 (M25)
Ast_{reqd}. = 602 mm^2/m width
$Ast_{provided}$ = 894 mm^2/m width > Ast_{reqd}; Safe.
Ast_{min} = 860 mm^2/m width

Shear

$pt_{provided}$ = 894 × 100 / (1000 × 717) = 0.12 %
τc = 0.29 for $p_{t\ provided}$; (IS 456:2000 Table 19)
Shear capacity = τc.b.d = 0.29 × 1000 × 717 / 1000 = 208 kN
V = 0 kN
Vu = 0 < τc.b.d; provide nominal shear reinforcement.

Shrinkage reinforcement at the top layer both ways:

Provide 16 diameter @ 400 mm c/c both ways (Ast = 503 mm^2 / m)
[Half of minimum reinforcement = 430 mm^2]

Side face reinforcement should be provided for pile caps having depth more than 600 mm.

Design of the pedestal shall be the same as the design of the column.

[Note for readers:

In this example, the pile cap slab is designed without any shear reinforcement. The depth of the pile cap slab has been increased up to the limit which can resist the applied shear by the shear strength of the concrete. Hence, the flexural strength of the pile cap slab is found much higher than the design moment even with minimum reinforcement.]

Strength design in ACI Code method

Reference code: ACI 318–19 Unit: SI metric; stress in Mpa

See Figure 5.36 Stress and strain diagram for the foundation slab.

Design parameters:

One-way slab (cantilever type)
fc′ =20 MPa bw = 1000 mm

fy = 415 MPa h = 800 mm (> span/10)
Es = 200000 MPa d′ = 75 mm cover
Bar diameter = 16 mm d = effective depth = 717 mm

Design load	*Factored load (*)*
Mx = 39 kNm	Mu long = 59 kNm
My = 59 kNm	Mu Tranv = 89 kNm
V = 131 kN	Vu = 197 kN

(*) Factored load should be determined according to ACI 318 – Table 5.3.1
Provide main bars:
Along the Y axis – 20 diameter @ 225 mm c/c (Ast = 1396 mm^2 /m width)
Along the X axis – 20 diameter @ 225 mm c/c (Ast = 1396 mm^2 /m width)
Minimum reinforcement: ACI 318–19 (7.6.1.1)
In flexure – As min = 0.0018 Ag = 0.0018 × 1000 × 800 = 1440 mm^2

To find out NA depth

Maximum usable strain at concrete compression fiber, ε_{cu} = 0.003
Net tensile strain at steel reinforcement,
ε_t = 0.005 [>= εt y + 0.003] ACI 318–19 (21.2.2.1)
ε_{ty} = fy / Es = 0.00208 tension reinforcement yielded.
Depth of the neutral axis from the top, c = ε_{cu}. d / (ε_{cu} + ε_t)
= 0.003 × 717 / (0.003 + 0.005) = 269 mm
a = β × c ACI 318–19 (21.2.2.4.1)
β = 0.85 ACI 318–19 (Table 22.2.2.4.3)
a = 0.85 × 269 = 229 mm

Along the Y axis:

Flexural reinforcement as a singly reinforced slab
Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)
Mu = 59 kNm tension-controlled
Required nominal strength, Mn = Mu/ ϕ = 65 kNm ϕ Mn > Mu
From stress block above, a = 229 mm c = 269 mm ε_{cu} = 0.003
C = 0.85 fc′. a. bw = 0.85 × 20 × 229 × 1000 / 1000 = 3893 kN

Mn_1 = C (d – a/2) = 3893 × (717 – 0.5 × 229) / 1000 = 2346 kNm > Mn; singly reinforced.
$As_{required}$ = Mn /fy. (d – a/2) = 65 × 1000000 / 415 × (717 – 0.5 × 229) = 260 mm^2
Reinforcement provided = 1396 mm^2 > $As_{required}$; Safe.

Transverse bar along the X axis

Flexural reinforcement as a singly reinforced slab
Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)
Mu = 89 kNm tension-controlled
Required nominal strength, Mn = Mu/ ϕ = 98 kNm ϕ Mn > Mu
C = 0.85 fc′. a. bw = 0.85 × 20 × 229 × 1000 / 1000 = 3893 kN

$Mn_1 = C\ (d - a/2) = 3893 \times (717 - 0.5 \times 229) / 1000 = 2346$ kNm > Mn; singly reinforced.

$As_{required} = Mn / fy. (d - a/2) = 98 \times 1000000 / 415 \times (717 - 0.5 \times 229) = 393\ mm^2$

Reinforcement provided = 1396 mm^2 > $As_{required}$; Safe.

Shear reinforcement

Strength reduction factor, $\phi = 0.75$ ACI 318–19 (Table 21.2.1)
Vu = 197 kN
Required nominal strength, $Vn = Vu / \phi = 262$ kN ϕ Vn > = Vu
Shear capacity = Vc ACI 318–19 (Table 22.5.5.1a)

$Vc = 0.17\ \lambda \sqrt{fc'}.\ bw.\ d = 0.17 \times 0.72 \times \sqrt{20} \times 1000 \times 717 / 1000 = 392$ kN.
$\lambda = \sqrt{2 / (1 + 0.004\ d)} = 0.72 <= 1$ ACI 318–19 (22.5.5.1.3)
Here, Vc > Vn
Shear reinforcement is not necessary.

Shrinkage reinforcement bars at the top layer ACI 318–19 (24.4.3.2)

Shrinkage reinforcement (0.0018Ag) should be distributed in two layers. There are tension reinforcement bars in the bottom layer, so provide half of minimum re-bars (0.0009Ag) at top.

Provide 16 diameter @ 240 c/c (Ast = 838 mm^2 /m) > 0.0009Ag = 720 mm^2 /m).

Design of the pedestal shall be the same as defined in ACI – 318.

As per ACI Code – ACI 318–19 in US customary unit

Design parameters:

See Figure 5.36 Stress and strain diagram for foundation slab.

fc′ = 3000 psi	fy = 60000 psi	Es = 29000000 psi
One-way slab	bw = 12 inch	h = 31 inch
Cover, d′ = 3 inch	d = 28 inch	

Design load (factored) ACI 318–19 (Table 5.3.1)
Mu Long = 13 kip-ft / ft
Mu Tranv = 20 kip-ft / ft
Vu = 13 kip /ft

Main bars at the bottom layer:

Along the Y axis	Bar no. # 6 @ 8 inch c/c	0.66 in^2 / ft
Along the X axis	Bar no. # 6 @ 8 inch c/c	0.66 in^2 / ft

Minimum reinforcement:
As min = $0.0018\ Ag = 0.0018 \times 12 \times 28 = 0.61\ in^2$ / ft ACI 318–19 (7.6.1.1)

Flexural reinforcement as a singly reinforced slab

Along the Y axis:

Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 13 kip-ft nominal strength = Mn; ϕ Mn >= Mu ACI 318–19 (9.5.1.1)
From Figure 5.36 – Stress and strain diagram for the slab above,
C = T 0.85 fc′ b a = As fy effective width b = bw = 12 inch

a = As fy / 0.85 fc′ b = 60000 / (0.85 × 3000 × 12) × As = 1.961 As
Ast provided = 0.66 in^2 /ft
So, a = 1.961 × 0.66 = 1.294 inch
ϕ Mn = ϕ As fy (d – a/2)

ϕ Mn = 0.9 × 0.66 × 60000 × (28.23 – 0.5 × 1.294) = 983 kip-inch = 82 kip-ft > Mu; safe.

Along the X axis:

Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 20 kip-ft nominal strength = Mn; ϕ Mn >= Mu ACI 318–19 (9.5.1.1)
From Figure 5.36 – Stress and strain diagram for the slab above,

C = T 0.85 fc′ b a = As fy effective width b = bw = 12 inch

a = As fy / 0.85 fc′ b = 60000 / (0.85 × 3000 × 12) × As = 1.961 As
Ast provided = 0.66 in^2 /ft
So, a = 1.961 × 0.66 = 1.294 inch
ϕ Mn = ϕ As fy (d – a/2)

ϕ Mn = 0.9 × 0.66 × 60000 × (28.23 – 0.5 × 1.294) = 983 kip-inch = 82 kip-ft > Mu; safe.

Shear check

ϕ Vn >= Vu ACI 318–19 (9.5.1.1)
Vn = Vc + Vs ACI 318–19 (9.5.3.1; 22.5.1.1)
Vc = 2 $\sqrt{fc'}$.bw.d ACI 318–19 (Table 22.5.1.1)
Required nominal strength =Vn
Strength reduction factor, $\phi = 0.75$ for shear ACI 318–19 (Table 21.2.1)
Vu = 13 kip

Shear capacity of the pile cap concrete slab

ϕ Vc = ϕ × 2 λ $\sqrt{fc'}$. bw. d = 0.75 × 2 × 0.72 × $\sqrt{(3000)}$ × 12 × 28.23 /1000 = 20 kips
$\lambda = \sqrt{[2/(1 + 0.1d)]} = 0.72 < 1$ ACI 318–19 (Table 22.5.5.1a)
ϕ Vc > Vu; shear reinforcement is not necessary. ACI 318–19 (22.5.5.1.3)
Maximum allowable shear in this C/S of the slab, Vu max = 111 kips ACI 318–19 (22.5.1.2)
Vu max <= ϕ (Vc + 8 $\sqrt{fc'}$. bw.d)
Vu < Vu max

Re-bars at top ACI 318–19 (24.4.3.2)

Shrinkage reinforcement (0.0018Ag = 0.61 inch^2) may be distributed in two layers. There are tension reinforcement bars in the bottom layer, so provide half of minimum re-bars (0.0009Ag = 0.31 inch^2) at the top.

Provide bar # 5 @ 10 inch centers (0.37 inch2) in both directions.

Design of the pedestal shall be the same as design of the column.

5.2.3 Pile cap for 5 pile group

This is a workout example of a pile cap resting on five piles in a group. Minimum distance between pile centers = 3 × diameter pile. The design procedure is shown in a step-by-step method.

5.2.3.1 Sketch

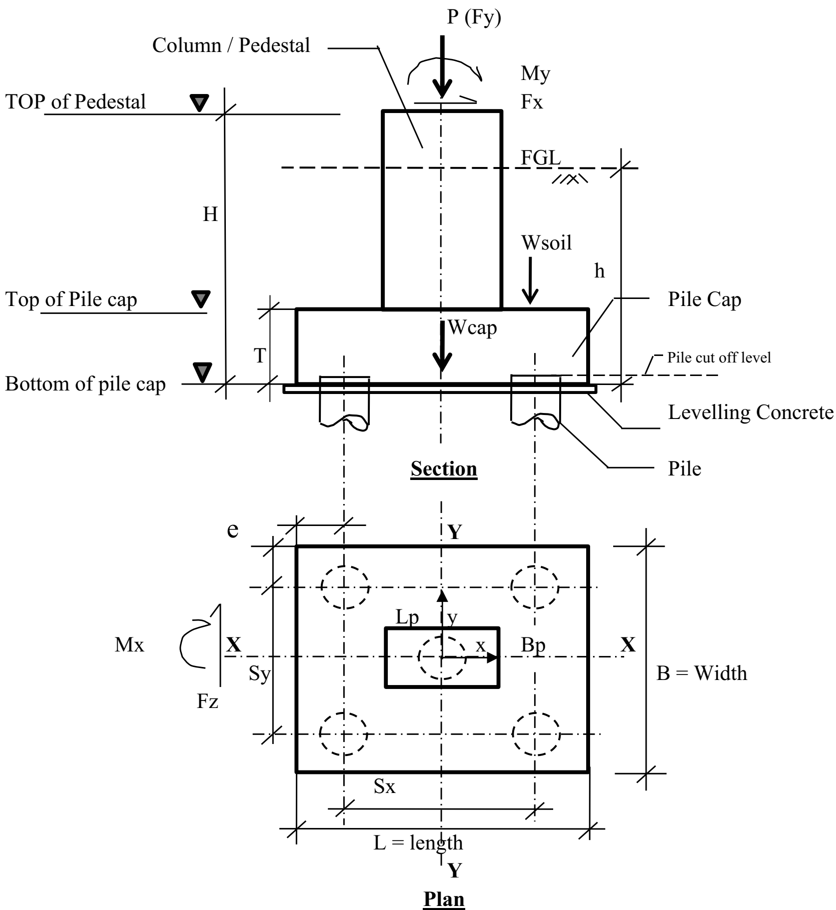

FIGURE 5.19 Pile cap – 5 pile group

Design parameters

Levels:

FGL = 0.3 M TOP = 1.2 M TOC = – 0.9 BOC = –1.9 M
GWL = – 2.5 M

Pile cap dimensions:

L = 2.7 m	B = 2.7 m	T = 1 m
Lp = 1.2 m	Bp = 0.9 m	e = 0.35 m
Sx = 2 m	Sy = 2 m	
h = 2.2 m	H = 3.1 m	
Cover = 75 mm	Maximum Bar diameter, ϕ diameter = 16 mm	

d, effective depth = 1–0.075 – 0.008 = 0.917 m.

Materials:

γconc = 25 kN/m^3 γsoil = 16 kN/m^3 Grade of Concrete = M25
fck = 25 MPa (cube strength) reinforcement steel = Fe415 fy = 415 MPa

Pile group detail:

Pile diameter, dp = 400 mm No. of piles, n = 5
Pile capacity: Axial = 400 kN (comp) tension = 200 kN lateral = 22 kN

Self-weight of the pile cap and pedestal, Wcap = [2.7 × 2.7 × 1 + 1.2 × 0.9 × (3.1 – 1)] × 25 = 239 kN
Weight of backfill soil above the pile cap, Wsoil = (2.7 × 2.7 – 1.2 × 0.9) × (2.2 – 1) × 16 = 119 kN

Pile group properties and load per pile:
Load per pile = P/n (+/–) Mx / Zy (+/–) My / Zx
where n = no. of piles

$$Zy = Iy / y = \Sigma y^2 / y$$
$$Zx = Ix / x = \Sigma x^2 / x$$

Maximum load/pile = $P/n + Mx.y / \Sigma y^2 + My.\ x / \Sigma x^2$
Minimum load/pile = $P/n - Mx.y / \Sigma y^2 - My.\ x / \Sigma x^2$

Now for the above pile group in Figure 5.39,

$x / \Sigma x^2 = (0.5 \times 2) / [4 \times (0.5 \times 2)^2] = 0.25$
$y / \Sigma y^2 = (0.5 \times 2) / [4 \times (0.5 \times 2)^2] = 0.25$

Let us design this pile group and pile cap, for the load combination case given below.

5.2.3.2 Pedestal/column load

TABLE 5.27
Design load

	Horizontal	Vertical	Horizontal	Moment			Load combination
Load case	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm	
1	12	1050	5	14	87	0	Dead + Wind

TABLE 5.28
Load on the pile group:

	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
	12	1050	5	14	87	
Wcap		239				
Wsoil		119				
Fz.H				15.50		
Fx.H					37.2	
Sum	**12**	**1408**	**5**	**30**	**124**	**0**
		P		**Mx**	**My**	

Note:

1. *Column loads are taken from a building frame analysis.*
2. *If GWL is above bottom of the pile cap level, then submerged weight of soil and concrete to be considered.*

Maximum load on piles

Axial compression = $P/n + Mx.y/\Sigma y^2 + My.x/\Sigma x^2$
$= 1408/5 + 30 \times 0.25 + 124 \times 0.25$
$= 282 + 7.5 + 31 = 320$ kN < 400 kN; Safe.
Tension/uplift = $P/n - Mx.y/\Sigma y^2 -$ **My**$.x/\Sigma x^2$
$= 1408/5 - 30 \times 0.25 - 124 \times 0.25$
$= 282 - 7.5 - 31 = 243$ kN < No Uplift.

Lateral = $\sqrt{(Fx^2 + Fz^2)} / n = \sqrt{(12^2 + 5^2)} / 5 = 13 / 5 = 2.60$ kN < 22kN; safe.

Strength design of pile cap

a) Along the Y axis:

Y = pile center distance from the face of the pedestal
X= critical shear plane from the face of the pedestal
d = effective depth of the pile cap

Y = (Sy − Bp)/2 = (2−0.9) / 2 = 0.55m
X = d/2 = 0.917 / 2 = 0.459 m
e_1 = distance between the pile center and critical shear plane
= Y − X = 0.55−0.459 = 0.09 m

Pile center line is outside of the critical shear plane but within dp/2 distance.

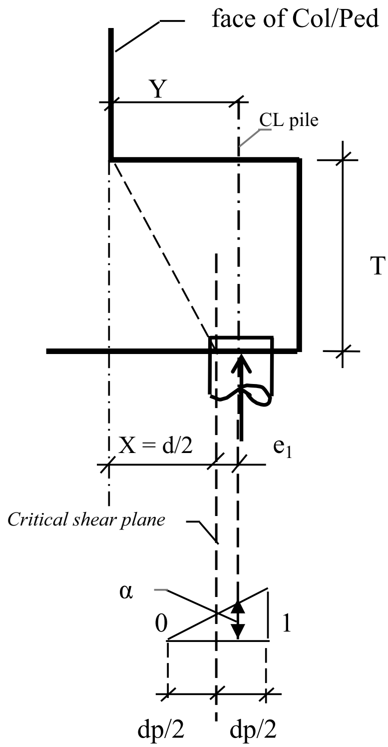

FIGURE 5.20 Part section along Y-Y

Pile load reduction factor, α: (IS 456: 2000 34.2.4.2)

α = 1 (full shear, when the pile center is dp/2 or more than dp/2 outside the critical section.)
α = 0 (no shear, when the pile center is located at dp/2 or more inside the critical section.)
α = 0 to 1 (for intermediate positions of the pile center, the portion of the pile reaction to be assumed as producing shear on the section shall be based on straight line interpolation between full value at dp/2 outside the section and zero value at dp/2 inside the section.)

Here,
e_1 = 0.09 m dp/2 = 0.4 / 2 = 0.2 m
Pile load reduction factor, α = (0.2 + 0.09) × (1 / 0.4) = 0.73
Effective No. of piles = 2
Total shear = 2 × 0.73 × 320 = 467 kN
Weight of the pile cap projecting outside the pedestal:
Width = 2.7 m projection = (2.7−0.9) / 2 =0.9 m
Self-weight of the pile cap projecting outside the face of the pedestal = 2.7 × 0.9 × 1 × 25 = 61 kN
Moment = (467 × 0.55 − 61 × 0.9 × 0.5) / 2.7 = 85 kNm/meter wide
Shear = (467−61) / 2.7 = 150 kN/meter wide

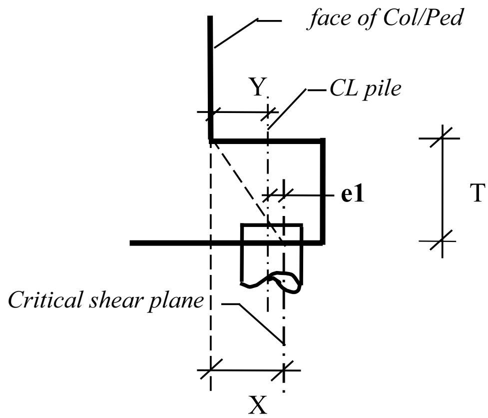

FIGURE 5.21 Part section at X-X

b) Along the X axis:

Y = (Sx-Lp)/2 = (2 − 1.2) / 2 = 0.4 m

X = d/2 = 0.917 / 2 = 0.459 m

e_1 = distance between the pile center and critical shear plane

e_1 = X − Y = 0.459−0.4 = 0.059 m

Center of the pile is inside the shear plane,

dp/2 = 0.2 m

Pile load factor, α = (0.2−0.059) × (1 / 0.4) = 0.35

Effective No. of piles = 2 Total shear = 2 × 0.35 × 320 = 224 kN

Weight of the pile cap projecting outside the pedestal:

Width = 2.7 m projection = (2.7−1.2) / 2 = 0.75 m

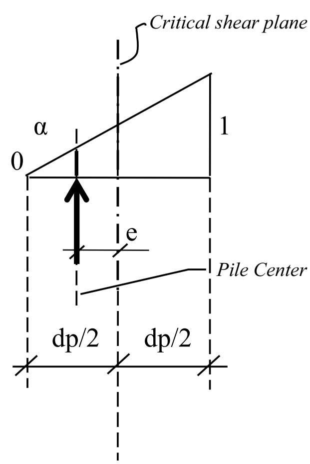

FIGURE 5.22 Reduction factor α

Self-weight of the pile cap projecting outside the face of the pedestal = 2.7 × 0.75× 1 × 25 = 51 kN

Moment = (224 × 0.4 − 51 × 0.5 × 0.75) / 2.7 = 26 kNm/meter wide

Shear = (224−51) / 2.7 = 64 kNm/meter wide

Reinforcement bars in the pile cap:

At the bottom layer,

Along the X axis – long bars – 16 mm diameter @ 175 mm c/c (Ast = 1149 mm^2/m)

Along the Y axis – transverse bars – 16 mm diameter @ 175 mm c/c (Ast = 1149 mm^2/m)

Strength design conforming to IS Code – IS: 456 and SP 16

a) Along the Y axis

Moment at the face of column = 85 kN/m   d prov = 917 mm   Load factor = 1.2

$Mu / bd^2 = 1.2 \times 85 \times 1000000 / (1000 \times 917^2) = 0.12$

pt = 0.084 %   for M25 (from SP 16 Table 3)

$Ast_{reqd.} = 0.084 \times 1000 \times 917 / 100 = 770$ mm²/m width

$Ast_{min} = 0.12 \times 1000 \times 917 / 100 = 1100$ mm²/m width

$Ast_{provided} = 1149$ mm²/m width; safe.

Shear:

$pt_{provided} = 1149 \times 100 / (1000 \times 917) = 0.13$ %

$\tau c = 0.29$ for pt provided;   (IS 456:2000 Table 19; SP 16 Table 61)

$\tau c.b.d = 0.29 \times 1000 \times 917 / 1000 = 266$ kN

V = 150 kN $Vu = 1.2 \times 150 = 180$ kN $< \tau c.b.d$; Safe, Shear reinforcement not required.

b) Along the X axis

Moment at the face of the column = 26 kN/m   d prov = 917 mm   Load factor = 1.2

$Mu / bd^2 = 1.2 \times 26 \times 1000000 / (1000 \times 917^2) = 0.04$

pt = 0.084 %   for M25   (from SP 16 Table 3)

$Ast_{reqd.} = 770$ mm²/m width

$Ast_{min} = 1100$ mm²/m width

$Ast_{provided} = 1149$ mm²/m width; safe.

Shear:

$pt_{provided} = 1149 \times 100 / (1000 \times 917) = 0.13$ %

$\tau c = 0.29$ for pt provided;   (IS 456:2000 Table 19; SP 16 Table 61)

$\tau c.b.d = 0.29 \times 1000 \times 917 / 1000 = 266$ kN

V = 64 kN $Vu = 1.2 \times 64 = 77$ kN $< \tau c.b.d$; safe, shear reinforcement not required.

Shrinkage reinforcement at top layer – both ways:

Provide 12 mm diameter @ 200 mm c/c (Ast = 566 mm²/m)
[Half of minimum reinforcement = 550 mm² / m]

Side face reinforcement should be provided for pile caps of depth more than 600 mm.

Design of the pedestal shall be the same as design of column.

[Note for readers: In this example, the pile cap slab is designed without any shear reinforcement. The depth of the pile cap slab has been increased up to the limit, which can resist the applied shear by the shear strength of the concrete. Hence, the flexural strength of the pile cap slab is found much higher than the design moment even with minimum reinforcement.]

Strength design conforming to ACI 318–19

Reference code: ACI 318–19 unit: SI metric; stress in Mpa

Design parameters:

One-way slab (cantilever type)

fc′ =20 MPa	bw = 1000 mm
fy = 415 MPa	h = 1000 mm
Es = 200000 MPa	d′ = 75 mm cover
Bar diameter = 20 mm	d = effective depth = 915 mm

Design load	*Factored load (*)*
Mx = 85 kNm	Mu long = 102 kNm
My = 26 kNm	Mu Tranv = 31 kNm
V = 150 kN	Vu = 180 kN

(*) Factored load should be determined according to ACI 318 – Table 5.3.1
Provide main bars:
Along the Y axis – 20 diameter @ 170 mm c/c (Ast = 1847 mm2 /m width)
Along the X axis – 20 diameter @ 170 mm c/c (Ast = 1847 mm2 /m width)
Minimum reinforcement: ACI 318–19 (7.6.1.1)
In flexure – As min = 0.0018 Ag = $0.0018 \times 1000 \times 1000 = 1800$ mm^2

See Figure 5.36 for stress and strain diagram for the foundation slab.

To find out NA depth

Maximum usable strain at concrete compression fiber, $\varepsilon_{cu} = 0.003$
Net tensile strain at steel reinforcement,
$\varepsilon_t = 0.005$ [>= εt y + 0.003] ACI 318–19 (21.2.2.1)
ε_{ty} = fy / Es = 0.00208 Tension reinforcement yielded.
Depth of the neutral axis from the top, c = ε_{cu}. d / ($\varepsilon_{cu} + \varepsilon_t$)
$= 0.003 \times 915 / (0.003 + 0.005) = 343$ mm
$a = \beta \times c$ ACI 318–19 (21.2.2.4.1)
$\beta = 0.85$ ACI 318–19 (Table 22.2.2.4.3)
$a = 0.85 \times 343 = 292$ mm

Re-bars along the Y axis

Flexural reinforcement as a singly reinforced slab
Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 102 kNm tension-controlled
Required nominal strength, Mn = Mu/ ϕ = 113 kNm ϕ Mn > Mu
From stress block above, a = 292 mm c = 343 mm $\varepsilon_{cu} = 0.003$
C = 0.85 fc′. a. bw = $0.85 \times 20 \times 292 \times 1000 / 1000 = 4964$ kN

Mn_1 = C (d – a/2) = $4964 \times (915 - 0.5 \times 292) / 1000 = 3817$ kNm > Mn; singly reinforced.

$As_{required} = Mn / fy. (d - a/2) = 113 \times 1000000 / 415 \times (915 - 0.5 \times 292) = 355\ mm^2$
Reinforcement provided = 1847 mm^2 > $As_{required}$; Safe.

Re-bars along the X axis

Flexural reinforcement as a singly reinforced slab
Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 31 kNm tension-controlled
Required nominal strength, $Mn = Mu/ \phi = 35$ kNm ϕ Mn > Mu
$C = 0.85\ fc'. a. bw = 0.85 \times 20 \times 292 \times 1000 / 1000 = 4964$ kN
$Mn_1 = C (d - a/2) = 4964 \times (915 - 0.5 \times 292) / 1000 = 3817$ kNm > Mn; singly reinforced.
$As_{required} = Mn / fy. (d - a/2) = 35 \times 1000000 / 415 \times (915 - 0.5 \times 292) = 109\ mm^2$
Reinforcement provided = 1847 mm^2 > $As_{required}$; Safe.

Shear reinforcement

Strength reduction factor, $\phi = 0.75$ ACI 318–19 (Table 21.2.1)
Vu = 180 kN
Required nominal strength, $Vn = Vu / \phi = 240$ kN ϕ Vn > = Vu
Shear capacity = Vc ACI 318–19 (Table 22.5.5.1a)

$Vc = 0.17\ \lambda \sqrt{fc'}. bw. d = 0.17 \times 0.655 \times \sqrt{20} \times 1000 \times 915 / 1000 = 456$ kN.
$\lambda = \sqrt{2 / (1 + 0.004\ d)} = 0.655 <= 1$ ACI 318–19 (22.5.5.1.3)
Here, Vc > Vn shear reinforcement not necessary

Maximum allowable shear in this cross section
of the slab, Vu_{max} ACI 318–19 (22.5.1.2)
$Vu_{max} = \phi (Vc + 0.66 \sqrt{fc}. bw. d) = 2026$ kN > Vu; Okay.

Shrinkage reinforcement bars at the top layer ACI 318–19 (24.4.3.2)

Shrinkage reinforcement (0.0018Ag) should be distributed in two layers. There are tension reinforcement bars in the bottom layer, so provide half of minimum re-bars (0.0009Ag) at the top.

Provide 16 diameter @ 200 c/c (Ast = 1006 mm^2 /m) > 0.0009Ag = 900 mm^2 /m.
Design of the pedestal shall be same as defined in ACI – 318.

5.2.3.3 Anchor bolt

The dimensional details of anchor bolts will be available in steel base plate drawings for the pile cap bearing steel column. However, the designer should check the concrete breakout strength or rupture of the pedestal over the embedded length of the anchor bolt.

The method of the concrete breakout strength determination is available in ACI 318R –14 – Building Code Requirements for Structural Concrete and Commentary.

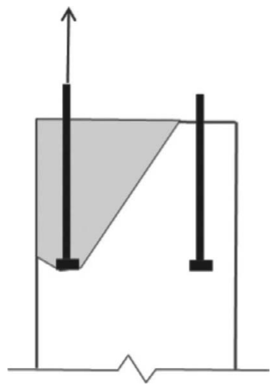

FIGURE 5.23 Concrete breakout shapes for the anchor bolt

5.2.4 Pile cap for 6 pile group

This workout example is for a pile cap resting on six piles in a group.

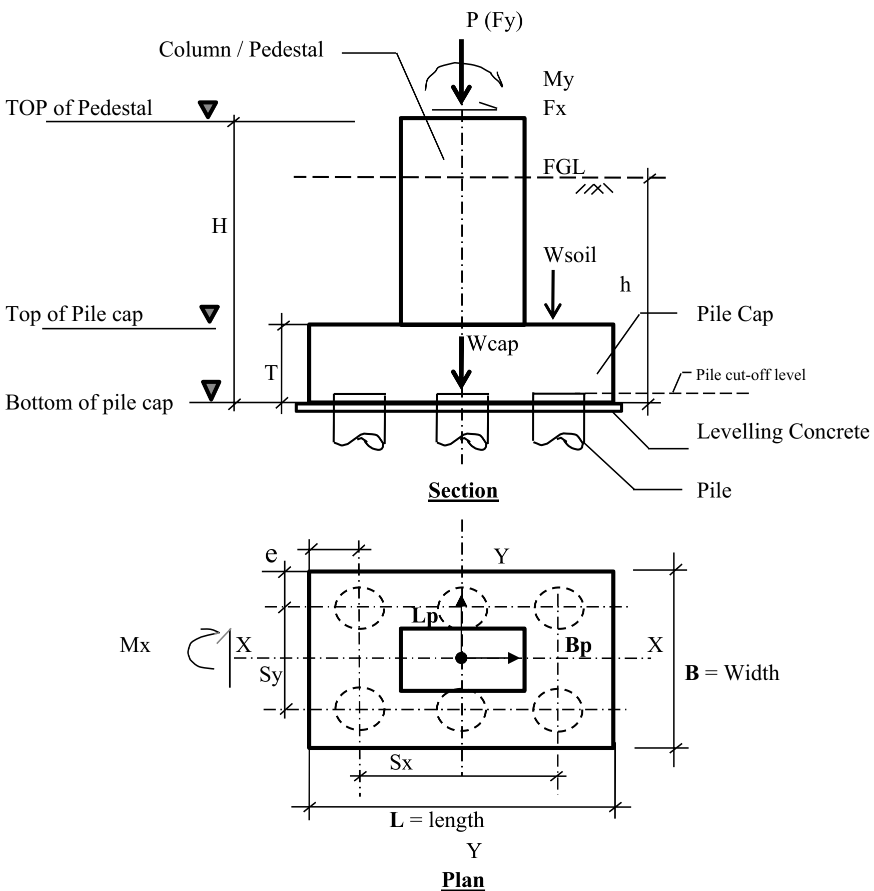

FIGURE 5.24 Pile cap – 6 pile group

Design parameters

Elevations	*pile cap dimensions:*
FGL = 0.3 M	length, L = 3.1 M
TOP = 1.2 M	width, B = 1.9 M
TOC = –1.3 M	thickness, T = 0.9 M
BOC = –2.2 M	pedestal length, Lp = 1.5 M
GWL = –3 M	width of the pedestal, Bp = 0.75 M
	Edge distance, e = 0.35 M

Height of pedestal above the bottom of the pile cap, H = 3.4 M
Depth of the pile cap below FGL, h = 2.5 M

Material

Unit weight of concrete, γconc = 25 kN/m^3
Unit weight of soil, γsoil = 16 kN/m^3

Concrete cover = 75 mm	maximum bar diameter, ϕ diameter = 16 mm
Grade of pile cap concrete = M25	reinforcement steel = Fe415

Pile capacity	*Pile group detail*
Axial = 450 kN	pile diameter, dp = 400 mm
Tension = 225 kN	Nos. (n) = 6
Lateral = 22 kN	minimum spacing of piles, S = 1.2 m (3 × dp)
	Sx = 2.40 m Sy = 1.20 m

Self-weight of the pile cap and pedestal,
Wcap = [3.1 × 1.9 × 0.9 + 1.5 × 0.75 × (3.4 − 0.9)] × 25 = 203 kN
Weight of backfill soil above the pile cap,
Wsoil = (3.1 × 1.9 − 1.5 × 0.75) × (2.5 − 0.9) × 16 = 122 kN

Pile group properties and load per pile:

Load per pile = P/n (+/–) Mx / Zy (+/–) My / Zx
where n = no. of piles

$Zy = Iy / y = \Sigma y^2 / y$

$Zx = Ix / x = \Sigma x^2 / x$

Maximum load/pile = $P/n + Mx.y / \Sigma y^2 + My.x / \Sigma x^2$
Minimum load/pile = $P/n - Mx.y / \Sigma y^2 - My.x / \Sigma x^2$
Now for the above pile group in Figure 5.39,

$x / \Sigma x^2 = (0.5 \times 2.4) / [4 \times (0.5 \times 2.4)^2] = 0.21$
$y / \Sigma y^2 = (0.5 \times 1.2) / [6 \times (0.5 \times 1.2)^2] = 0.28$

Let us design this pile group and pile cap, for one load combination case.

5.2.4.1 Pedestal/column load

TABLE 5.29
Design load

	Horizontal	Vertical	Horizontal	Moment			Load combination
Load case	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm	
1	30	1200	25	250	150	30	Dead + Wind

TABLE 5.30
Load on pile group:

	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
	30	1200	25	250	150	
Wcap		203				
Wsoil		122				
Fz.H				85		
Fx.H					102	
Sum	**30**	**1525**	**25**	**335**	**252**	**0**
		P		**Mx**	**My**	

Note:
1. *Column loads are taken from a building frame analysis.*
2. *If GWL is above the bottom of the pile cap level, then submerged weight of soil and concrete to be considered.*

Maximum load on piles

Axial compression = $P/n + Mx.y/\Sigma y^2 + My.x/\Sigma x^2$
$= 1525 / 6 + 335 \times 0.28 + 252 \times 0.21$
$= 254 + 93.8 + 52.92 = 401$ kN < 450 kN; Safe.

Tension/uplift = $P/n - Mx.y/\Sigma y^2 - \mathbf{My}.x/\Sigma x^2$
$= 1525 / 6 - 335 \times 0.28 - 252 \times 0.21$
$= 254.17 - 93.8 - 52.92 = 107$ kN < No Uplift.

Lateral = $\sqrt{(Fx^2 + Fz^2)} / n = \sqrt{(30^2 + 25^2)} / 6 = 39.05 / 6 = 6.51$ kN < 22kN; safe.

Strength design of the pile cap

a) Along the Y axis:

Y = Pile center distance from the face of the pedestal

X = critical shear plane from the face of the pedestal
d = effective depth of the pile cap

Y = (Sy − Bp)/2 = (1.2−0.75) / 2 = 0.225 m
X = d/2 = 0.817 / 2 = 0.409 m

e_1 = distance between the pile center and critical shear plane
= X − Y = 0.409−0.225 = 0.184 m
Pile center line is inside the critical shear plane
But less than dp/2 away from the critical shear plane.

Pile load reduction factor, α*:* (IS 456: 2000 34.2.4.2)
Pile load reduction factor, α = (0.2 − 0.184) × (1 / 0.4) = 0.04
Effective No. of piles = 3
Total shear = 3 × 0.04× 401 = 48 kN

Weight of the pile cap projecting outside the pedestal:
Width = 3.1 m Projection = (1.9−0.75) / 2 = 0.575 m

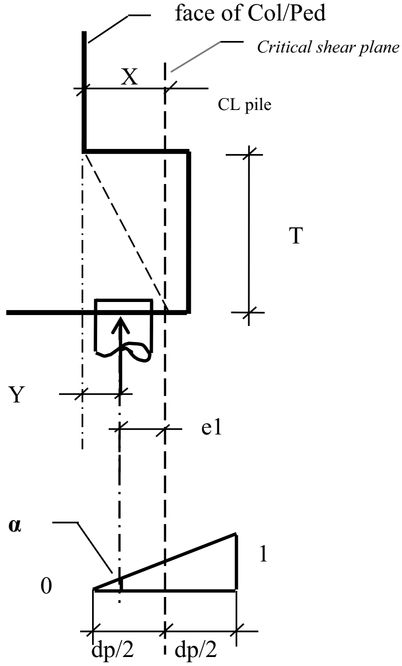

FIGURE 5.25 Part section along Y-Y

Self-weight of the pile cap projecting outside the face of the pedestal = 3.1 × 0.575 × 0.9 × 25 = 40 kN

Moment = (3 × 401 × 0.225 − 40 × 0.575 × 0.5) / 3.1 = 84 kNm/meter wide (*)
Shear = (48−40) / 3.1 = 3 kN/meter wide
(*) conservative approach

c) Along the X axis:

Y = (Sx − Lp) /2 = (2.4 − 1.5) / 2 = 0.45 m
X = d/2 = 0.817 / 2 = 0.409 m
e_1 = distance between the pile center and critical shear plane
e_1 = Y − X = 0.45−0.409 = 0.041 m

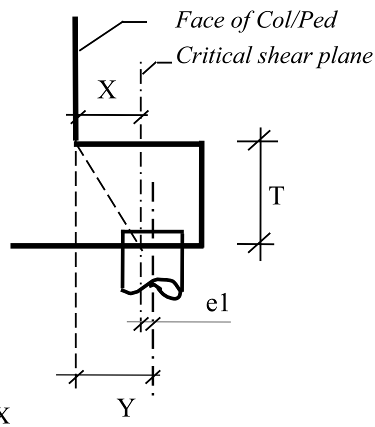

FIGURE 5.26 Part section at X-X

Center of the pile is outside the critical shear plane,

dp/2 = 0.2 m

Pile load factor, $\alpha = (0.2 + 0.041) \times (1 / 0.4) = 0.60$

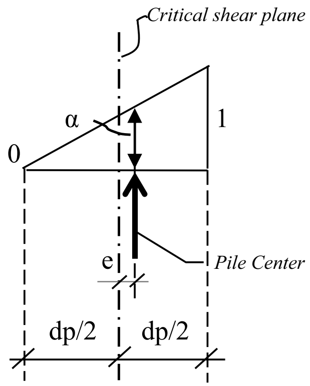

FIGURE 5.27 Reduction factor α

Effective No. of piles = 2 Total shear = 2 × 0.6 × 401 = 481 kN

Weight of the pile cap projecting outside the pedestal:

Width = 1.9 m Projection = (3.1 − 1.5) / 2 = 0.8 m

Self-weight of the pile cap projecting outside the face of the pedestal = 1.9 × 0.8 × 0.9 × 25 = 34 kN

Moment = (481 × 0.45 − 34 × 0.5 × 0.8) / 1.9 = 107 kNm/meter wide

Shear = (481–34) / 1.9 = 235 kNm/meter wide

Reinforcement bars in the pile cap:

At the bottom layer

Along the X axis – long bars − 16 mm diameter @ 100 mm c/c (Ast = 2011 mm^2/m)

Along the Y axis – transverse bars − 16 mm diameter @ 150 mm c/c (Ast = 1341 mm^2/m)

Strength design conforming to IS Code – IS: 456 and SP 16

c) Along the Y axis

Moment at the face of the col = 84 kN/m d prov = 817 mm Load factor = 1.2

$Mu/bd^2 = 1.2 \times 85 \times 1000000 / (1000 \times 817^2) = 0.15$

pt = 0.084 % for M25 (from SP 16 Table 3)

$Ast_{reqd.} = 0.084 \times 1000 \times 817 / 100 = 686\ mm^2/m$ width

$Ast_{min} = 0.12 \times 1000 \times 817 / 100 = 980\ mm^2/m$ width

$Ast_{provided} = 1341\ mm^2/m$ width; safe.

Shear:

$pt_{provided} = 1341 \times 100 / (1000 \times 817) = 0.16\ \%$

$\tau c = 0.297$ for pt provided; (IS 456:2000 Table 19; SP 16 Table 61)

$\tau c.b.d = 0.297 \times 1000 \times 817 / 1000 = 243$ kN

V = 3 kN Vu = 1.2 × 3 = 3.6 kN < τc.b.d; safe, shear reinforcement not required.

d) Along the X axis

Moment at the face of column = 107 kN/m d prov = 817 mm Load factor = 1.2
$Mu /bd^2 = 1.2 \times 107 \times 1000000 / (1000 \times 817^2) = 0.19$
pt = 0.084 % for M25 (from SP 16 Table 3)
$Ast_{reqd.} = 686\ mm^2/m$ width
$Ast_{min} = 980\ mm^2/m$ width
$Ast_{provided} = 2011\ mm^2/m$ width; Safe.

Shear:

$pt_{provided} = 2011 \times 100 / (1000 \times 817) = 0.25\ \%$
$\tau c = 0.36$ for pt provided; (IS 456:2000 Table 19; SP 16 Table 61)
$\tau c.b.d = 0.36 \times 1000 \times 817 / 1000 = 294$ kN
V = 235 kN $Vu = 1.2 \times 235 = 282$ kN < τc.b.d; safe, shear reinforcement not required.

Shrinkage reinforcement at the top layer – both ways:

Provide 12 mm diameter @ 200 mm c/c (Ast = 566 mm^2/m)
[Half of minimum reinforcement = 490 mm^2 / m]

Side face reinforcement should be provided for pile caps of depth more than 600 mm.
Design of the pedestal shall be the same as design of column.

Strength design conforming to ACI 318–19

Reference code: ACI 318–19 unit: SI metric; stress in Mpa

Design parameters:

One-way slab (cantilever type)

fc′ =20 MPa	bw = 1000 mm
fy = 415 MPa	h = 900 mm
Es = 200000 MPa	d′ = 75 mm cover
Bar diameter = 16 mm	d = effective depth = 817 mm

Design load	*factored load (*)*
Mx = 84 kNm	Mu long = 101 kNm
My = 107 kNm	Mu Tranv = 128 kNm
V = 235 kN	Vu = 282 kN

(*) Factored load should be determined according to ACI 318 – Table 5.3.1
Provide main bars:
Along the Y axis – 16 diameter @ 100 mm c/c (Ast = 2010 mm^2 /m width)
Along the X axis – 16 diameter @ 100 mm c/c (Ast = 2010 mm^2 /m width)
Minimum reinforcement: ACI 318–19 (7.6.1.1)
In flexure – As min = 0.0018 Ag = $0.0018 \times 1000 \times 900 = 1620\ mm^2$
See Figure 5.36 for stress and strain diagram of the foundation slab.

To find out NA depth

Maximum usable strain at concrete compression fiber, $\varepsilon_{cu} = 0.003$
Net tensile strain at steel reinforcement,
$\varepsilon_t = 0.005$ [>= εt y + 0.003] ACI 318–19 (21.2.2.1)
$\varepsilon_{ty} = fy / Es = 0.00208$ tension reinforcement yielded.
Depth of the neutral axis from the top, $c = \varepsilon_{cu}.\ d / (\varepsilon_{cu} + \varepsilon_t)$
$= 0.003 \times 817 / (0.003 + 0.005) = 306$ mm
$a = \beta \times c$ ACI 318–19 (21.2.2.4.1)
$\beta = 0.85$ACI 318–19 (Table 22.2.2.4.3)
$a = 0.85 \times 306 = 260$ mm

Re-bars along the Y axis

Flexural reinforcement as a singly reinforced slab
Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 101 kNm tension-controlled
Required nominal strength, $Mn = Mu/\ \phi = 112$ kNm ϕ Mn > Mu
From the stress block above, a = 260 mm c = 306 mm $\varepsilon_{cu} = 0.003$
$C = 0.85\ fc'.\ a.\ bw = 0.85 \times 20 \times 260 \times 1000 / 1000 = 4420$ kN
$Mn_1 = C\ (d - a/2) = 4420 \times (817 - 0.5 \times 260) / 1000 = 3037$ kNm > Mn; singly reinforced.
$As_{\ required} = Mn\ /fy.\ (d - a/2) = 112 \times 1000000 / 415 \times (817 - 0.5 \times 260) = 393\ mm^2$
Reinforcement provided = 2010 mm^2 > $As_{\ required}$; Safe.

Re-bars along the X axis

Flexural reinforcement as a singly reinforced slab
Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 128 kNm tension-controlled
Required nominal strength, $Mn = Mu/\ \phi = 143$ kNm ϕ Mn > Mu
$C = 0.85\ fc'.\ a.\ bw = 0.85 \times 20 \times 260 \times 1000 / 1000 = 4420$ kN
$Mn_1 = C\ (d - a/2) = 4420 \times (817 - 0.5 \times 260) / 1000 = 3037$ kNm > Mn; singly reinforced.
$As_{\ required} = Mn\ /fy.\ (d - a/2) = 143 \times 1000000 / 415 \times (817 - 0.5 \times 260) = 500\ mm^2$
Reinforcement provided = 2010 mm^2 > $As_{\ required}$; safe.

Shear reinforcement
Strength reduction factor, $\phi = 0.75$ ACI 318–19 (Table 21.2.1)
Vu = 282 kN
Required nominal strength, $Vn = Vu /\ \phi = 376$ kN ϕ Vn > = Vu
Shear capacity = Vc ACI 318–19 (Table 22.5.5.1a)
$Vc = 0.17\ \lambda \sqrt{fc'}.\ bw.\ d = 0.17 \times 0.68 \times \sqrt{20} \times 1000 \times 817 / 1000 = 425$ kN.
$\lambda = \sqrt{2 / (1 + 0.004\ d)} = 0.68 <= 1$ ACI 318–19 (22.5.5.1.3)
Here, Vc > Vn shear reinforcement not necessary

Maximum allowable shear in this cross section of the slab, Vu_{max} — ACI 318–19 (22.5.1.2)

$Vu_{max} = \phi\ (Vc + 0.66\ \sqrt{fc}.\ bw.\ d) = 1809\ kN > Vu$; okay.

Shrinkage reinforcement bars at the top layer ACI 318–19 (24.4.3.2)

Shrinkage reinforcement (0.0018Ag) should be distributed in two layers. There are tension reinforcement bars in the bottom layer, so provide half of minimum rebars (0.0009Ag) at the top.

Provide 16 diameter @ 200 c/c (Ast = 1006 mm^2 /m) > 0.0009Ag = 810 mm^2 /m.
Design of the pedestal shall be the same as defined in ACI – 318.

5.2.4.2 Anchor bolt

The dimensional details of anchor bolts will be available in steel base plate drawings for pi\le cap bearing steel column. However, the designer should check the concrete breakout strength or rupture of the pedestal over the embedded length of anchor bolt conforming to ACI 318R –14 – Building Code Requirements for Structural Concrete and Commentary.

5.2.5 Pile cap for 16 pile group

This is a workout example of a pile cap resting on 16 pile group, which is commonly used in the powerhouse, coal mill building and similar heavily loaded columns. The pile diameter is 500 mm; minimum distance between pile centers = 3 × diameter of pile.

The piles are spaced at 3×diameter of piles.

Design parameters

Elevations	Pile cap dimensions:
FGL = 0.3 M	length, L = 6.2 M
TOP = 1.2 M	width, B = 5.3 M
TOC = −1.6 M	thickness, T = 1.4 M
BOC = –3.0 M	pedestal length, Lp = 2.5 M
GWL = −4.0 M	width of the pedestal, Bp = 2.2 M
	Edge distance, e = 0.4 M

Height of the pedestal above the bottom of the pile cap, H = 4.2 M
Depth of the pile cap below FGL, h = 3.3 M

Material

Unit weight of concrete, $\gamma conc = 25\ kN/m^3$
Unit weight of soil, $\gamma soil = 16\ kN/m^3$

Concrete cover = 75 mm	maximum bar diameter, ϕ diameter = 28 mm
Grade of pile cap concrete = M25	reinforcement steel = Fe415

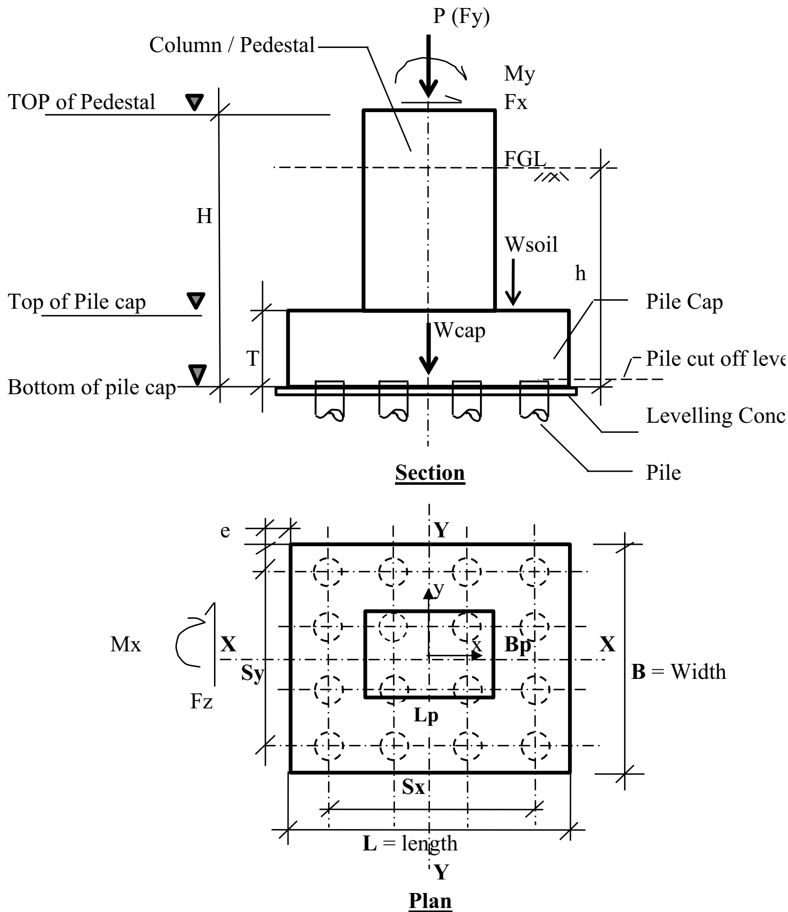

FIGURE 5.28 Pile cap for 16 pile group

Pile capacity	*Pile group detail*
Axial = 900 kN	Pile diameter, dp = 500 mm
Tension = 450 kN	Nos. (n) = 16
Lateral = 60 kN	Minimum spacing of piles,
	S = 1.5 m (3 × dp)
	Sx = 3 × 1.8 = 5.40 m
	Sy = 3 × 1.5 = 4.5 m

Self-weight of the pile cap and pedestal,

$$\text{Wcap} = [6.2 \times 5.3 \times 1.4 + 2.5 \times 2.2 \times (4.2{-}1.4)] \times 25 = 1535 \text{ kN}$$

Weight of backfill soil above the pile cap,

$$\text{Wsoil} = (6.2 \times 5.3{-}2.5 \times 2.2) \times (3.3{-}1.4) \times 16 = 832 \text{ kN}$$

Pile group properties and load per pile:

Load per pile = P/n (+/–) Mx / Zy (+/–) My / Zx
where n = No. of piles
$Zy = Iy / y = \Sigma y^2 / y$
$Zx = Ix / x = \Sigma x^2 / x$
Maximum load/pile = $P/n + Mx.y / \Sigma y^2 + My. x / \Sigma x^2$
Minimum load/pile = $P/n - Mx.y / \Sigma y^2 - My. x / \Sigma x^2$
Now for above the pile group in Figure 5.48,
$x / \Sigma x^2 = (0.5 \times 5.4) / [8 \times (0.5 \times 1.8)^2 + 8 \times (1.5 \times 1.8)^2] = 0.042$
$y / \Sigma y^2 = (0.5 \times 4.5) / [8 \times (0.5 \times 1.5)^2 + 8 \times (1.5 \times 1.5)^2] = 0.050$
Let us design this pile group and pile cap, for one load combination case.

5.2.5.1 Pedestal/column load

TABLE 5.31
Design load

	Horizontal	Vertical	Horizontal	Moment			Load combination
Load case	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm	
1	185	7000	80	0	650		Dead + Roof Live load + Floor Live load + Temperature load

TABLE 5.32
Load on pile group:

	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
	185	9595	80	0	650	
Wcap		1535				
Wsoil		832				
Fz.H				336		
Fx.H					777	
Sum	**185**	**11962** **P**	**80**	**336** **Mx**	**1427** **My**	**0**

Note:

1. *Column loads are taken from a building frame analysis.*
2. *If GWL is above the bottom of the pile cap level, then submerged weight of soil and concrete to be considered.*

Maximum load on piles

Axial compression = P/n + Mx.y/Σy² + My.x/Σx²
= 11962 / 16 + 336 × 0.05 + 1427 × 0.042
= 747.6 + 16.8 + 59.93 = 824 kN < 900 kN; Safe.

Tension/uplift = P/n − Mx.y/Σy² − **My**.x/Σx²
= 11962 / 16−336 × 0.05 − 1427 × 0.042
= 747.6 − 16.8 − 59.93 = 671 kN No Uplift.

Lateral = $\sqrt{(Fx^2 + Fz^2)}$ / n = $\sqrt{(185^2 + 80^2)}$ / 16 = 202 / 16 = 12.6 kN < 60 kN; safe.

5.2.5.2 Strength design of the pile cap

a) Along the Y axis:

Y = pile center distance from the face of the pedestal
X = critical shear plane from the face of the pedestal
d = effective depth of the pile cap

Row 1

Y_1 = (Sy − Bp)/2 = (4.5−2.2) / 2 = 1.15 m
Center of piles are outside the shear plane.

Row 2

Y_2 = 0.5 × 2.2−0.5 × 1.5 = 0.35 m
Pile center is inside the face of the pedestal and the critical shear plane.
X = d/2 = 1.31 / 2 = 0.655 m
e = X + Y_2 = 0.655 + 0.355 = 1.005m > dp /2 (= 0.5 / 2 = 0.25m)
Hence, these piles will not produce any shear.

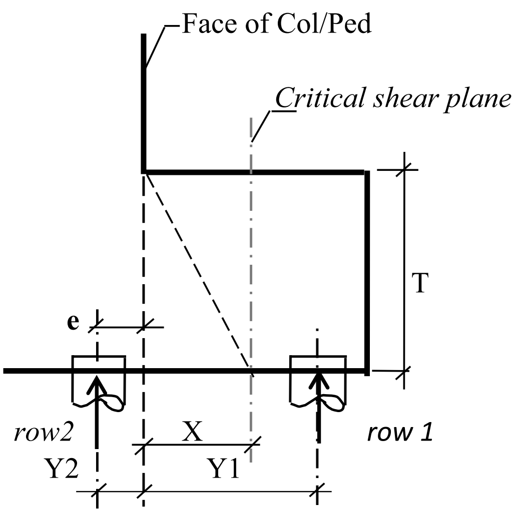

FIGURE 5.29 Part section at Y-Y

Pile load reduction factor, α:

Row 1 – pile outside the critical shear plane and more than dp/2 away so will generate full shear.
α = 1
Row 2 – pile center is inside the pedestal and more than dp/2 away from the critical shear plane; hence, will not produce any shear.

Effective No. of piles outside the critical shear plane = 43

Total shear = 4 × 1× 824 = 3296 kN

Weight of the pile cap projecting outside the pedestal:
Width = 6.2 m projection = (5.3–2.2) / 2 = 1.55 m
Self-weight of the pile cap projecting outside the face of the pedestal = 6.2 × 1.55 × 1.4 × 25 = 336 kN
Moment = (3296 × 1.15 – 336 × 0.5 × 1.55) / 6.2 = 569 kNm/meter wide
Shear = (3296–336) / 6.2 = 477 kN/meter wide

b) Along the X axis:

Row 1 is outside the face of the pedestal

Y_1 = (Sx – Lp) /2 = (5.4 – 2.5) / 2 = 1.45 m
X = d/2 = 1.31 / 2 = 0.655 m
e = distance between the pile center and critical shear plane
e = Y_1 – X = 1.45–0.655 = 0.795 m > dp/2
These piles will produce full shear.

Row 2 – Inside the face of the pedestal

Y_2 = distance from the center of pile cap = 0.5 × 1.8 = 0.9 m
X_2 = critical shear plane from the center of the pile cap
= 0.5 × 1.31 + 0.5 × 2.5 = 1.905 m
X_2 – Y_2 = =1.905–0.9 = 1.005 > dp/2.
Hence, these piles will not produce any shear on the pile cap.

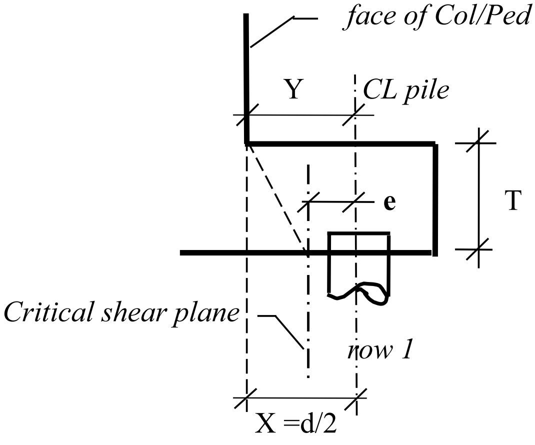

FIGURE 5.30 Part section at X X

Effective No. of piles = 4 total shear = 4 × 824 = 3296 kN
Weight of the pile cap projecting outside the pedestal:
Width = 5.3 m projection = (6.2 – 2.5) / 2 = 1.85 m
Self-weight of the pile cap projecting outside the face of the pedestal = 5.3 × 1.85 × 1.4 × 25 = 343 kN
Moment = (3296 × 1.45 – 343 × 0.5 × 1.85) / 5.3 = 842 kNm/meter wide
Shear = (3296–343) / 5.3 = 557 kNm/meter wide

Reinforcement bars in the pile cap:

At the bottom layer

Along the X axis – long bars – 28 mm diameter @ 175 mm c/c (Ast = 3520 mm^2/m)
Along the Y axis – transverse bars – 28 mm diameter @ 175 mm c/c (Ast = 3520 mm^2/m)

Strength design conforming to IS Code – IS: 456 and SP 16

e) Along the Y axis

Moment at the face of column = 569 kN/m   d prov = 1310 mm

Load factor = 1.5
$Mu /bd^2 = 1.5 \times 569 \times 1000000 / (1000 \times 1310^2) = 0.50$
pt = 0.142 %   for M25 (from SP 16 Table 3)
$Ast_{reqd.} = 0.142 \times 1000 \times 1310 / 100 = 1860$ mm^2/m width
$Ast_{min} = 0.12 \times 1000 \times 1310 / 100 = 1572$ mm^2/m width
$Ast_{provided} = 3520$ mm^2/m width; safe.

Shear
$pt_{provided} = 3520 \times 100 / (1000 \times 1310) = 0.27$ %
$\tau c = 0.36$ for pt provided; (IS 456:2000 Table 19; SP 16 Table 61)
$\tau c.b.d = 0.36 \times 1000 \times 1310 / 1000 = 472$ kN
V = 477 kN $Vu = 1.5 \times 477 = 716$ kN   > $\tau c.b.d$; provide shear reinforcement.
Let us provide vertical stirrups in the form of single links at equal spacing on both sides
- 20 mm diameter @ 400 mm centers (single leg)

Vus = shear capacity of vertical stirrups
Asv = cross-sectional area of the stirrup leg = 314 mm^2
sv = spacing of stirrups = 400 mm c/c
d = effective depth of the pile cap = 1310 mm
fy = 415 MPa
$Vus = 0.87\ fy\ Asv\ d / sv = [0.87 \times 415 \times 314 \times 1310 / 400] / 1000 = 371$ kN
Shear capacity = $Vus + \tau c.b.d = 371 + 472 = 843$ kN > Vu; safe.

f) Along the X axis
Moment at the face of column = 842 kN/m   d prov = 1310 mm
Load factor = 1.5
$Mu /bd^2 = 1.5 \times 842 \times 1000000 / (1000 \times 1310^2) = 0.74$
pt = 0.216 %   for M25 (from SP 16 Table 3)
$Ast_{reqd.} = 2830$ mm^2/m width
$Ast_{min} = 1572$ mm^2/m width
$Ast_{provided} = 3520$ mm^2/m width; safe.

Shear:

$pt_{provided} = 3520 \times 100 / (1000 \times 1310) = 0.27$ %
$\tau c = 0.36$ for pt provided; (IS 456:2000 Table 19; SP 16 Table 61)
$\tau c.b.d = 0.36 \times 1000 \times 1310 / 1000 = 472$ kN

V = 557 kN   Vu = 1.5 × 557 = 836 kN   > τc.b.d; provide shear reinforcement.

Let us provide vertical stirrups in the form of single links at equal spacing on both sides

- 20 mm diameter @ 400 mm centers (single leg)

Vus = shear capacity of vertical stirrups
Asv = cross-sectional area of the stirrup leg = 314 mm^2
sv = spacing of stirrups = 400 mm c/c
d = effective depth of the pile cap = 1310 mm
fy = 415 MPa
Vus = 0.87 fy Asv d / sv = [0.87 × 415 × 314 × 1310 / 400] / 1000 = 371 kN
Shear capacity = Vus + τc.b.d = 371 + 472 = 843 kN > Vu; Safe.

Shrinkage reinforcement at the top layer – both ways:

Provide 20 mm diameter @ 400 mm c/c (Ast = 786 mm^2/m)
[Half of minimum reinforcement = 786 mm^2 / m]

Side face reinforcement should be provided for pile caps of depth more than 600 mm.
Design of the pedestal shall be the same as design of column.

Strength design conforming to ACI 318–19

Reference code: ACI 318–19   unit: SI metric; stress in Mpa

Design parameters:

One-way slab (cantilever type)

fc′ =20 MPa	bw = 1000 mm
fy = 415 MPa	h = 1400 mm
Es = 200000 MPa	d′ = 75 mm cover
Bar diameter = 28 mm	d = effective depth = 1311 mm

Design load	*Factored load (*)*
Mx = 569 kNm	Mu long = 854 kNm
My = 842 kNm	Mu Tranv = 1263 kNm
V = 557 kN	Vu = 836 kN

(*) Factored loads conform to ACI 318 – Table 5.3.1

Provide main bars:
Along the Y axis – 28 diameter @ 175 mm c/c   (Ast = 3517 mm^2 /m width)
Along the X axis – 28 diameter @ 175 mm c/c   (Ast = 3517 mm^2 /m width)
Minimum reinforcement: ACI 318–19 (7.6.1.1)
In flexure –   As min = 0.0018   Ag = 0.0018 × 1000 × 1400 = 2520 mm^2
See Figure 5.36 for stress and strain diagram of a foundation slab.
To find out NA depth
Maximum usable strain at concrete compression fiber, ε_{cu} = 0.003
Net tensile strain at steel reinforcement,
ε_t = 0.005 [>= εt y + 0.003]   ACI 318–19 (21.2.2.1)

ε_{ty} = fy / Es = 0.002 Tension reinforcement yielded.
Depth of the neutral axis from the top, c = ε_{cu}. d / (ε_{cu} + ε_t)
= 0.003 × 1311 / (0.003 + 0.005) = 492 mm
a = β × c ACI 318–19 (21.2.2.4.1)
β = 0.85ACI 318–19 (Table 22.2.2.4.3)
a = 0.85 × 492 = 418 mm

Re-bars along the Y axis

Flexural reinforcement as a singly reinforced slab
Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)
Mu = 854 kNm tension-controlled
Required nominal strength, Mn = Mu/ ϕ = 948 kNm ϕ Mn > Mu
From the stress block above, a = 418 mm c = 492 mm ε_{cu} = 0.003
C = 0.85 fc′. a. bw = 0.85 × 20 × 418 × 1000 / 1000 = 7106 kN
Mn_1 = C (d − a/2) = 7106 × (1311 − 0.5 × 418) / 1000 = 7831 kNm > Mn; singly reinforced.
As $_{required}$ = Mn /fy. (d − a/2) = 984 × 1000000 / 415 × (1311 − 0.5 × 418)
= 2074 mm^2
Reinforcement provided = 3517 mm^2 > As $_{required}$; Safe.

Re-bars along the X axis

Flexural reinforcement as a singly reinforced slab
Strength reduction factor, ϕ = 0.9 for momentACI 318–19 (Table 21.2.1)
Mu = 1263 kNm tension-controlled
Required nominal strength, Mn = Mu/ ϕ = 1403 kNm ϕ Mn > Mu
C = 0.85 fc′. a. bw = 0.85 × 20 × 418 × 1000 / 1000 = 7106 kN
Mn_1 = C (d − a/2) = 7106 × (1311 − 0.5 × 418) / 1000 = 7831 kNm > Mn; singly reinforced.
As $_{required}$ = Mn /fy. (d − a/2) = 1403 × 1000000 / 415 × (1311 − 0.5 × 418)
= 3069 mm^2
Reinforcement provided = 3517 mm^2 > As $_{required}$; safe.

Shear reinforcement

Strength reduction factor, ϕ = 0.75 ACI 318–19 (Table 21.2.1)
Vu = 836 kN
Required nominal strength, Vn = Vu / ϕ = 1114 kN ϕ Vn > = Vu
Shear capacity = Vc ACI 318–19 (Table 22.5.5.1a)
Vc = 0.17 λ √fc′. bw. d = 0.17 × 0.57× √20 × 1000 × 1311 / 1000 = 564 kN.
λ = √ 2 / (1 + 0.004 d) = 0.57 < = 1 ACI 318–19 (22.5.5.1.3)
Here, Vc < Vn shear reinforcement will be necessary.
Let us provide vertical stirrups in the form of single links at equal spacing on both sides.

- 25 mm diameter bars @ 450 mm centers

Vs = Av fyt d /s ACI 318–19 (22.5.8.5.3)

s = longitudinal spacing
Av = effective area of bars
Vs= shear capacity of vertical stirrups
Av = effective area of all stirrup bar legs = $3.14 \times 25^2 / 4 = 491\ mm^2$
s = longitudinal spacing of shear reinforcement = 450 mm c/c
d = effective depth of the pile cap = 1311 mm
fyt = 415 MPa
Vs = Av fyt d / s = $491 \times 415 \times 1311 / 450 = 594$ kN
Shear capacity, Vc +Vs = 564 + 594 = 1158 kN > Vn; Safe — ACI 318–19 (22.5.8.1)

Maximum allowable shear in this cross section of the slab, Vu_{max} — ACI 318–19 (22.5.1.2)
$Vu_{max} = \phi\ (Vc + 0.66 \sqrt{fc}.\ bw.\ d) = 2903$ kN > Vu; Okay.

Shrinkage reinforcement bars at the top layer — ACI 318–19 (24.4.3.2)

Shrinkage reinforcement (0.0018Ag) should be distributed in two layers. There are tension reinforcement bars at the bottom layer, so provide half of minimum rebars (0.0009Ag) at the top.

Provide 20 diameter @ 2225 c/c (Ast = 1397 mm^2 /m) > = 1260 mm^2 /m; okay.
[1/2 of minimum reinforcement (0.0018Ag) = 2520 / 2 = 1260 mm^2]
Design of the pedestal shall be the same as defined in ACI – 318.
Side face reinforcement should be provided in the pile cap.

5.2.5.3 Anchor bolt

The dimensional details of anchor bolts will be available in the steel base plate drawing for the pile cap bearing steel column. However, the designer should check the concrete breakout strength or rupture of the pedestal over the embedded length of the anchor bolt conforming to ACI 318R -14 – Building Code Requirements for Structural Concrete and Commentary.

5.2.6 Pile group patterns

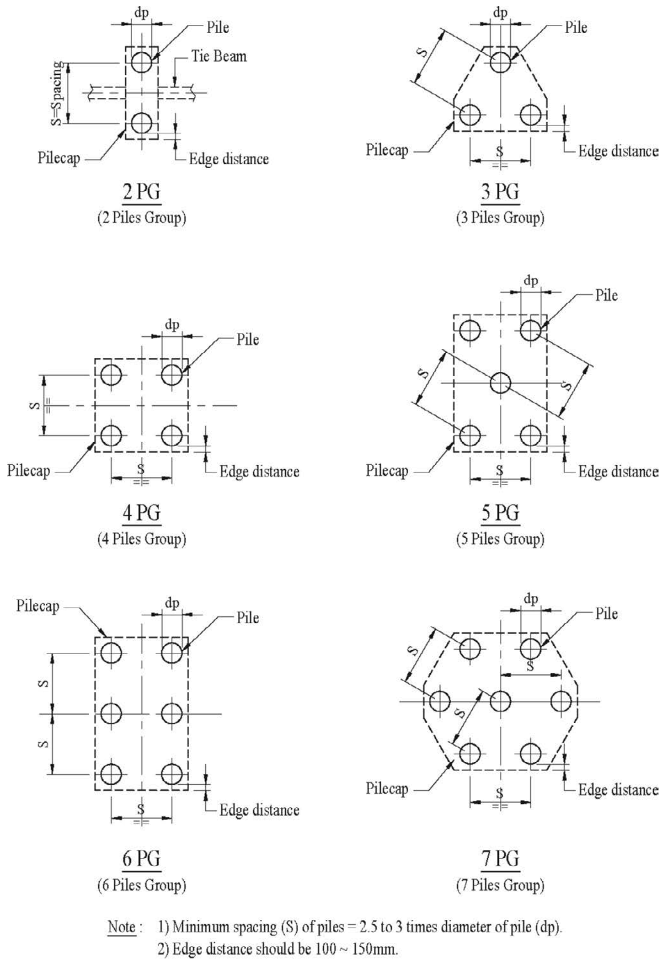

FIGURE 5.31 Standard pile group (PG) patterns – Sheet 1

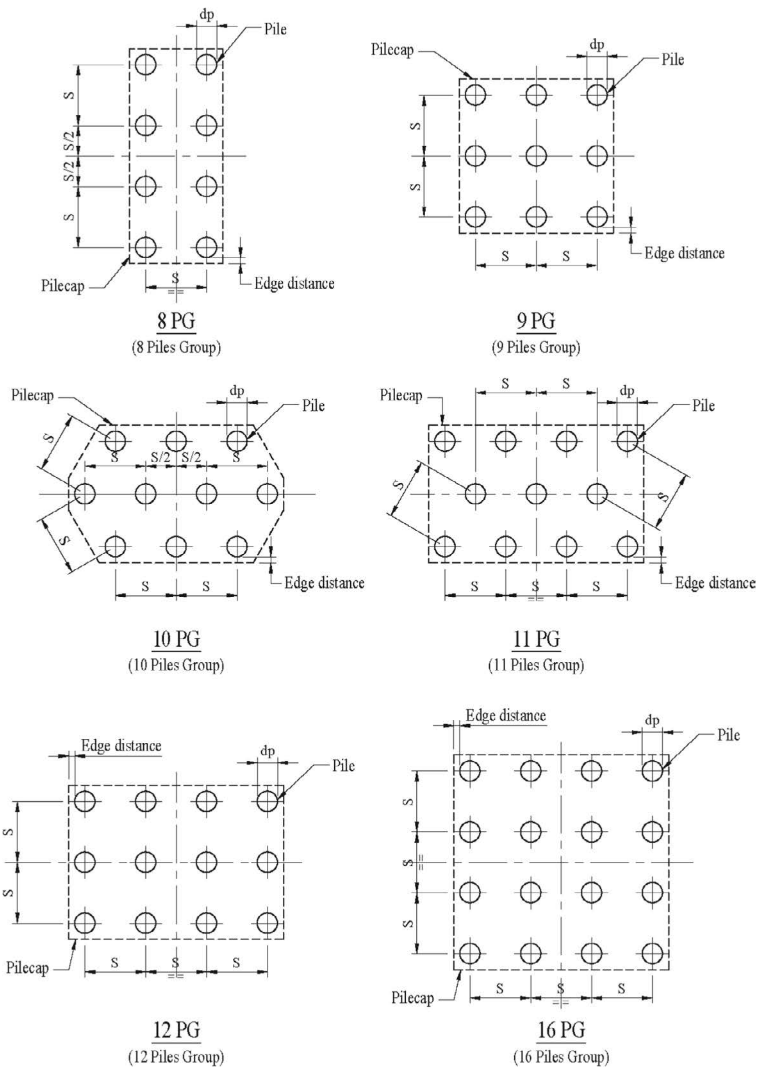

FIGURE 5.32 Standard pile group (PG) patterns – Sheet 2

5.3 DESIGN OF ISOLATED FOOTING

5.3.1 Flat footing

This is a workout example of a flat base isolated spread footing resting on soil.

5.3.3.1 Sketch

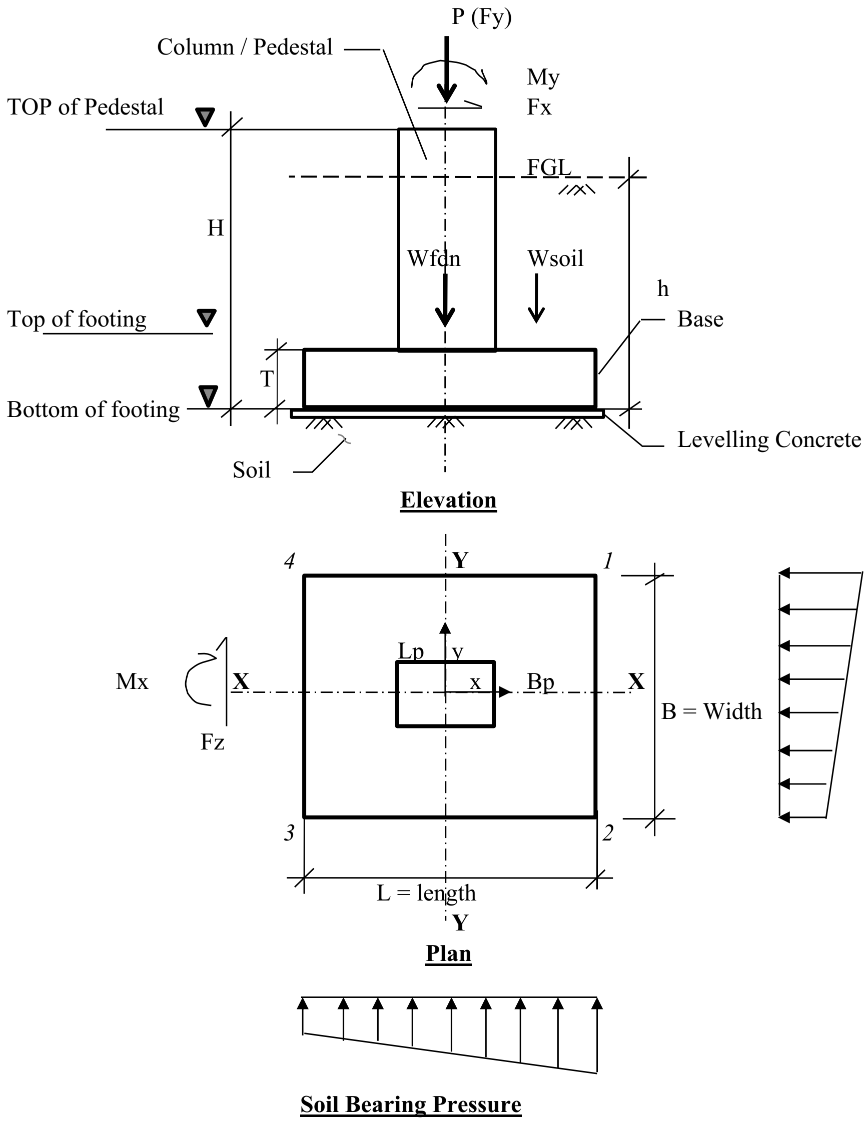

FIGURE 5.33 Isolated footing – flat-based

Input data

SBP = 250 kN/m^2	Net safe bearing pressure
γsur = 5 kN/m^2	Ground surcharge
FGL = 0.3 M	Finish-grade level
TOP = 1.2 M	Top of the pedestal
TOF = −1.35 M	Top of footing
BOC = −1.8 M	Bottom of footing
GWL = −2.5 M	Groundwater level (*)
γconc = 25 kN/m^3	Unit weight of concrete
γsoil = 16 kN/m^3	Unit weight of soil
Cover = 75 mm	Concrete cover
ϕ diameter = 12 mm	Maximum diameter of re-bars

(*) If GWL is above the bottom of footing, then the submerged weight of soil and concrete shall be considered for computation self-weight.

Footing dimensions:

Length, L = 2.2 m Width, B = 2 m thickness, T = 0.45 m
Effective depth, d = (450 − 75−0.5 ×12) /1000 = 0.369 m
Pedestal length, Lp = 0.6 m Width of the pedestal, Bp = 0.45 m
Depth of the footing below FGL, h = 2.1 m
Height of the pedestal above the bottom of the footing, H = 3 m

Materials:

Grade of concrete = M25 fck = 25 MPa (cube strength)
Reinforcement steel = Fe415 fy = 415 MPa

Calculations:

Self-weight of the base and pedestal, Wfdn
= [2.2 × 2 × 0.45 + 0.6 × 0.45 × (3 − 0.45)] × 25 = 67 kN
Weight of backfill soil above the footing slab, Wsoil
= (2.2 × 2−0.6 × 0.45) × (2.1 − 0.45) × 16 = 109 kN
Surcharge load, Wsur = (2.2 × 2 − 0.6 × 0.45) × 5 = 20.65 kN

Let us design this foundation for one load combination case.

TABLE 5.33
Column load

Load case	Horizontal	Vertical	Horizontal	Moment		Load combination
	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	
1	12	500	5	10	80	Dead + Wind

TABLE 5.34
Foundation load at the base

Item	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm
Column load	12	500	5	10	80
Wfdn		67			
Wsoil		109			
Wsur		21			
Fz.H				15.00	
Fx.H					36
Sum	**12**	**696**	**5**	**25**	**116**
		P		**Mx**	**My**

Soil bearing pressure:

Area at base = 2.2 × 2 = 4.4 m^2
Zx = 2.2 × 2^2 / 6 = 1.47 m^3 Zy = 2 × 2.2^2 / 6 = 1.61 m^3
P/A = 696 / 4.4 = 158 kN/m^2
Mx / Zx = 25 / 1.47 = 17 kN/m^2 My / Zy = 116 / 1.61 = 72 kN/m^2

Maximum GBP (gross bearing pressure) at corners:

Corner 1: 158 + 17 + 72 = 247 kN/m^2 < safe gross bearing pressure; safe.
Corner 2: 158 − 17 + 72 = 213 kN/m^2
Corner 3: 158 − 17–72 = 69 kN/m^2 no uplift at the corner.
Corner 4: 158 + 17–72 = 103 kN/m^2

Average soil bearing pressure = (247 + 213 + 69 + 103) / 4 = 158 kN/m^2 < gross bearing pressure; safe.
Gross soil bearing pressure, GBP = net SBP + γsoil × h = 250 + 16 × 2.1 = 285 kN/m^2
Self-weight of the footing slab = 0.45 × 25 = 15 kN/m^2
Weight of backfill above the footing slab = (2.1–0.45) × 16 = 26 kN/m^2
Net upward pressure = 158–15–26 = 117 kN/m^2 (for strength design of a slab.)

Stability check

Uplift – no uplift; so overturning check is not required.
Sliding – sliding force = $\sqrt{(12^2 + 5^2)}$ = 13 kN
Sliding-resistant capacity = μ × P = 0.33 × 696 = 230 kN
Factor of safety = 230 / 13 = 18 > 1.5; safe.

[For granular soil, the coefficient of friction μ is equal to tan ϕ, ϕ being the angle of friction of soil. The sliding stability on cohesive soil, the adhesion between the base slab and the soil is assumed to be equal to the cohesive strength of soil (c) multiplied by footing area.]

5.3.2.1 Strength design of footing slab

Along the Y axis:

Critical shear plane from the face of the pedestal = X
X = d = 0.369 m (T − cover − ϕdia/2)
Projection of the footing slab outside the pedestal,
Y = (2−0.45) / 2 = 0.775 m
Y − X = 0.775−0.369 = 0.406 m

Width of the slab = 2.2 m
Net upward soil pressure = 117 kN/m^2
Moment = (2.2 × 0.775 × 117) × (0.775 / 2) = 77 kNm for full width and 77 / 2.2 = 35 kNm per meter wide.
Shear = 2.2 × 0.406 × 117= 105 kN for full width and 105 /2.2 = 48 kN/meter wide.

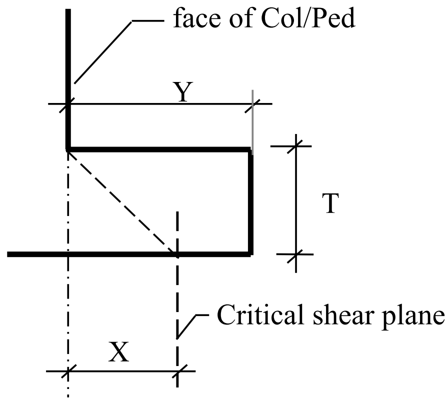

FIGURE 5.34 Part section at Y-Y

Along the X axis:

Y = (2.2−0.6) / 2 = 0.8 m X = d = 0.369 m
Y − X = 0.8−0.369 = 0.431 m
Width of slab = 2 m
Net upward soil pressure = 117 kN/m^2
Moment = (2 × 0.8 × 117) × (0.8 / 2) = 75 kNm for full width and 75 / 2 = 38 kN/m meter width
Shear = = 2 × 0.431 × 117= 101 kN for full width and 101 / 2 = 51 kN/meter wide.

Provide reinforcement bars:

At the bottom layer
Along the X axis – long bars: 12 mm diameter @ 250 mm c/c (Ast = 453 mm^2 /m)
Along the Z axis – transverse bars: 12 mm diameter @ 250 mm c/c (Ast = 453 mm^2 /m)

Strength design in conformity with IS Code – IS: 456 and SP 16

g) Along the Y axis:

Moment at the face of column = 35 kN/m d prov = 369 mm Load factor = 1.2
Mu /bd^2 = 1.2 × 35 × 1000000 / (1000 × 369^2) = 0.31
pt = 0.1 % for M25 (from SP 16 Table 3)
Ast $_{reqd.}$ = 0.1 × 1000 × 369 / 100 = 369 mm^2/m width
Ast $_{min}$ = 443 mm^2/m width (0.12 %)
Ast $_{provided}$ = 453 mm^2/m width; > Ast $_{reqd}$; safe.

Shear:

pt $_{provided}$ = 453 × 100 / (1000 × 369) = 0.12 %
τc = 0.29 for pt provided; (IS 456:2000 Table 19; SP 16 Table 61)
τc.b.d = 0.29 × 1000 × 369 / 1000 = 107 kN
V = 48 kN Vu = 1.2 × 48 = 58 kN < τc.b.d; safe, shear reinforcement not required.

h) Along the X axis:

Moment at the face of the column d prov = 369 mm Load factor = 1.2
= 38 kN/m
Mu /bd^2 = 1.2 × 38 × 1000000 / (1000 × 369^2) = 0.33
pt = 0.099 % for M25 (from SP 16 Table 3)
Ast $_{reqd.}$ = 365mm^2/m width
Ast $_{min}$ = 443 mm^2/m width
Ast $_{provided}$ = 453 mm^2/m width; > Ast $_{reqd}$; safe.

Shear:

pt $_{provided}$ = 453 × 100 / (1000 × 369) = 0.12 %
τc = 0.29 for pt provided; (IS 456:2000 Table 19; SP 16 Table 61)
τc.b.d = 0.29 × 1000 × 369 / 1000 = 107 kN
V = 51 kN Vu = 1.2 × 51 = 61 kN < τc.b.d; safe, shear reinforcement not required.

Shrinkage reinforcement at the top layer – both ways:

Provide 12 mm diameter @ 400 mm c/c (Ast = 283 mm^2 /m) > 221 mm^2 /m; okay.
Half of minimum reinforcement = 221 mm^2 /m

Design of the pedestal will be the same as the design of the column (IS 456 and SP16).

Strength design in conformity with the ACI Code – ACI 318–19 in US customary unit

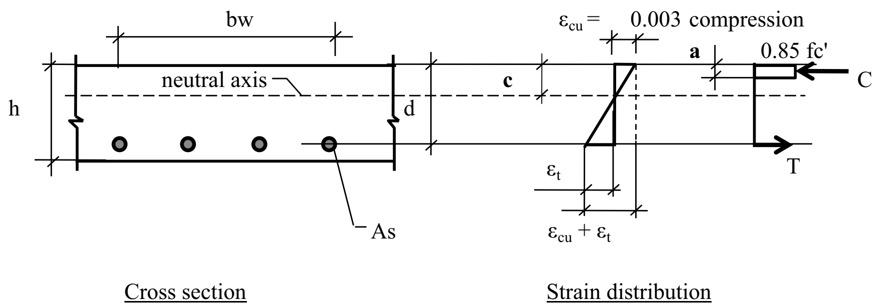

FIGURE 5.35 Stress-strain distribution diagram of a slab

Design parameters:

$fc' = 3000$ psi $fy = 60000$ psi $Es = 29000000$ psi
One-way slab $bw = 12$ inch $h = 18$ inch
Cover, $d' = 3$ inch $d = 15$ inch
Design load (factored)   ACI 318–19 (Table 5.3.1)
Mu Long = 9 kip-ft / ft
Mu Tranv = 10 kip-ft / ft
Vu = 4 kip /ft

Main bars at the bottom layer:

Along the Y axis bottom layer – bar no. # 4 @ 10 inch c/c   0.24 in^2 / ft
Top layer – bar no. # 4 @ 10 inch c/c   0.24 in^2 / ft
Total = 0.48 in^2 / ft > Minimum reinforcement
Along the X axis bottom layer – bar no. # 4 @ 10 inch c/c   0.24 in^2 / ft
Top layer – bar no. # 4 @ 10 inch c/c   0.24 in^2 / ft
Total = 0.48 in^2 / ft > minimum reinforcement

Minimum reinforcement:
As minimum = 0.0018 Ag = $0.0018 \times 12 \times 14.53 = 0.31$ in^2 / ft   ACI 318–19 (7.6.1.1)

Flexural reinforcement as a singly reinforced slab

Along the Y axis:

Strength reduction factor, $\phi = 0.9$ for moment   ACI 318–19 (Table 21.2.1)
Mu = 9 kip-ft   nominal strength = Mn; ϕ Mn > = Mu   ACI 318–19 (9.5.1.1)
From Figure 5.55 – stress and strain diagram for a slab above,

C = T   0.85 fc′ b a = As fy   effective width b = bw = 12 inch

a = As fy / 0.85 fc′ b = 60000 / (0.85 × 3000 × 12) × As = 1.961 As
Ast provided = 0.24 in^2 /ft
So, a = 1.961 × 0.24 = 0.471 inch
ϕ Mn = ϕ As fy (d – a/2)
ϕ Mn = 0.9 × 0.24 × 60000 × (14.53 – 0.5 × 0.471) = 185 kip-inch = 15 kip-ft > Mu; safe.

Along the X axis:

Strength reduction factor, $\phi = 0.9$ for moment   ACI 318–19 (Table 21.2.1)
Mu = 10 kip-ft   nominal strength = Mn; ϕ Mn > = Mu   ACI 318–19 (9.5.1.1)
From Figure 5.55 – stress and strain diagram for the slab above,

C = T   0.85 fc′ b a = As fy   effective width b = bw = 12 inch
a = As fy / 0.85 fc′ b = 60000 / (0.85 × 3000 × 12) × As = 1.961 As

Ast provided = 0.24 in^2 /ft
So, a = 1.961 × 0.24 = 0.471 inch

ϕ Mn = ϕ As fy (d − a/2)

ϕ Mn = 0.9 × 0.24 × 60000 × (14.53 − 0.5 × 0.471) = 185 kip-inch = 15 kip-ft > Mu; safe.

Shear:

ϕ Vn > = Vu ACI 318–19 (9.5.1.1)
Vn = Vc + Vs ACI 318–19 (9.5.3.1; 22.5.1.1)
Vc = 2 $\sqrt{fc'}$.bw.d ACI 318–19 (Table 22.5.1.1)
Required nominal strength =Vn
Strength reduction factor, ϕ = 0.75 for shear ACI 318–19 (Table 21.2.1)
Vu = 4 kip

Shear capacity = Vc

ϕ Vc = ϕ × 2 λ $\sqrt{fc'}$. bw. d = 0.75 × 2 × 0.9 × $\sqrt{(3000)}$ × 12 × 14.53 /1000 = 13 kips
$\lambda = \sqrt{[2/(1 + 0.1d)]} = 0.90 < 1$ ACI 318–19 (Table 22.5.5.1a)
ϕ Vc > Vu; shear reinforcement is not necessary. ACI 318–19 (22.5.5.1.3)

Maximum allowable shear in this C/S of the slab, Vu max = 57 kips ACI 318–19 (22.5.1.2)
Vu max <= ϕ (Vc + 8 $\sqrt{fc'}$. bw.d)
Vu < Vu max
Re-bars at top (along Y and X axes) ACI 318–19 (24.4.3.2)
Shrinkage reinforcement (0.0018Ag = 0.31 inch^2) may be distributed in two layers. There are tension reinforcement bars at the bottom layer, so provide half of minimum re-bars (0.0009Ag = 0.16 inch^2) at the top.
Provided re-bars: Bar # 4 @ 10 inch centers (0.24 inch^2) in both directions at the top layer.
Hence, safe.
Design of the pedestal shall be the same as the design of the column.

Strength design in accordance with ACI Code – ACI 318–19 SI unit

Refer to *Figure 5.55.*

Design parameters:

fc′ = 20 MPa fy = 415 MPa Es = 200000 MPa
One-way slab bw = 1000 mm h = 450 mm
Cover, d′ = 75 mm d = 369 mm bar diameter = 12 mm
Design load (factored) ACI 318–19 (Table 5.3.1)
Mu Long = 42 kNm
Mu Tranv = 46 kNm
Vu = 61 kN

Main bars at the bottom layer:

Along the Y axis bottom layer: 12 mm diameter@ 250 mm c/c (Ast = 453 mm^2 /m width.)
Top layer: 12 mm diameter @ 250 mm c/c (Ast = 453 mm^2 /m width.)
Total = 906 mm^2 /m width > minimum reinforcement
Along the X axis bottom layer: 12 mm diameter @ 250 mm c/c (Ast = 453 mm^2 /m width.)
Top layer: 12 mm diameter @ 250 mm c/c (Ast = 453 mm^2 /m width)
Total = 906 mm^2 /m width > minimum reinforcement
Minimum reinforcement:

As min = 0.0018 Ag = 0.0018 × 1000 × 450 = 810 mm^2 /m ACI 318–19 (7.6.1.1)

Along the Y axis:

Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)
Mu = 42 kNm nominal strength = Mn; ϕ Mn > = Mu ACI 318–19 (9.5.1.1)
From Figure 5.55 – stress and strain diagram for the slab above,
C = T 0.85 fc′ bw a = as fy effective width b = bw = 1000 mm
a = As fy / 0.85 fc′ b = 453 ×415/ (0.85 × 20 × 1000) = 11.1 mm
ϕ Mn = ϕ As fy (d − a/2)
ϕ Mn = 0.9 × 453 × 415 × (369 − 0.5 × 11.1) = 61 kNm = > Mu; Safe.

Along the X axis:

Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)
Mu = 46 kNm nominal strength = Mn; ϕ Mn > = Mu ACI 318–19 (9.5.1.1)
From Figure 5.55 – stress and strain diagram for a slab above,
C = T0.85 fc′ bw a = As fy effective width b = bw = 1000 mm
a = As fy / 0.85 fc′ b = 453 × 415 / (0.85 × 20 × 1000) = 11.1 mm
ϕ Mn = ϕ As fy (d − a/2)
ϕ Mn = 0.9 × 453 × 415 × (369 − 0.5 × 11.1) = 61 kNm = > Mu; Safe.

Shear:

Strength reduction factor, ϕ = 0.75 for shear ACI 318–19 (Table 21.2.1)
Vu = 61 kN required nominal strength, V_n = Vu / ϕ = 82 kN
Vc = 0.17 λ $\sqrt{fc'}$. bw. d = 0.17 × 0.9 × $\sqrt{20}$ × 1000 × 369 / 1000 = 252 kN
$\lambda = \sqrt{[2/(1 + 0.004d)]} = 0.90 < 1$ ACI 318–19 (Table 22.5.5.1a)
Vc > Vu; shear reinforcement is not necessary. ACI 318–19 (22.5.5.1.3)
Maximum allowable shear in this C/S of a slab, ACI 318–19 (22.5.1.2)
Vu max = 817 kN
Vu max <= ϕ (Vc + 0.66 $\sqrt{fc'}$. bw.d)
Vu < Vu max
Re-bars at top (along Y and X axes) ACI 318–19 (24.4.3.2)

Shrinkage reinforcement (0.0018Ag = 810 mm^2) may be distributed in two layers. There are tension reinforcement bars at the bottom layer, so provide half of minimum re-bars (0.0009Ag = 405 mm^2) at the top.

Provided re-bars: 12 mm diameter @ 250 mm C/C (Ast = 453 mm^2) in both directions at the top layer.

Hence, safe.

Design of the pedestal shall be the same as the design of the column.

5.3.2 Sloped footing

This is a workout example of a sloped footing resting on soil.

5.3.2.1 Sketch

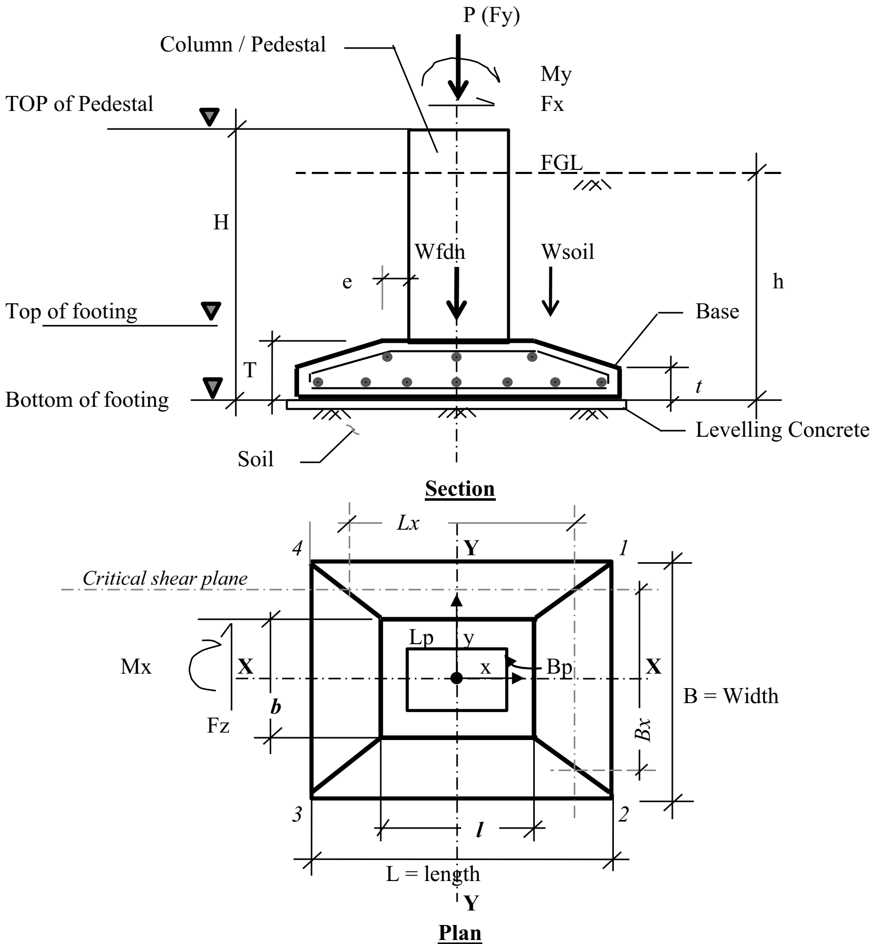

FIGURE 5.36 Isolated slope footing

Input data

SBP = 250 kN/m^2	net safe bearing pressure
γsur = 5 kN/m^2	ground surcharge
FGL = 0.3 M	finish-grade level
TOP = 1.2 M	top of the pedestal
TOF = −1.35 M	top of footing
BOC = −1.8 M	bottom of footing
GWL = −2.5 M	groundwater level (*)
γconc = 25 kN/m^3	unit weight of concrete
γsoil = 16 kN/m^3	unit weight of soil
Cover = 75 mm	concrete cover
ϕ diameter = 12 mm	maximum diameter of re-bars

(*)If GWL is the above bottom of footing, then submerged weight of soil and concrete shall be considered for computation self-weight.

Footing dimensions:

Length, L = 2 m *l* = 0.9 m width, B = 2 m b = 0.75 m e = 0.15 m
Thickness, T = 0.45 m *t* = 0.15 m
Effective depth, d = (450 − 75−0.5 ×12) /1000 = 0.369 m
Pedestal length, Lp = 0.6 m width of the pedestal, Bp = 0.45 m
Depth of footing below FGL, h = 2.1 m
Height of the pedestal above the bottom of footing, H = 3 m

Materials:

Grade of concrete = M25 fck = 25 MPa (cube strength)
Reinforcement steel = Fe415 fy = 415 MPa

Calculations:

Self-weight of the pedestal = 0.6 × 0.45 × (3 − 0.45)) × 25 = 17.21 kN
Self-weight of the sloped base = (0.45 / 3) × ($2^2 + 0.9^2$ + 2 × 0.9) × 25 = 24.79 kN
Total weight, Wfdn = 17.21 + 24.79 = 42 kN

Weight of backfill soil above the footing slab, Wsoil
= [(2 × 2 × 2.1) − (24.79 / 25)] × 16 = 119 kN
Surcharge load, Wsur = (2 × 2−0.6 × 0.45) × 5 = 18.65 kN

Let us design this foundation, for one load combination case.

TABLE 5.35
Column load

Load case	Horizontal	Vertical	Horizontal	Moment		Load combination
	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	
1	12	500	5	10	80	Dead + Wind

TABLE 5.36
Foundation load at base

Item	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm
Column load	12	500	5	10	80
Wfdn		42			
Wsoil		119			
Wsur		19			
Fz.H				15.00	
Fx.H					36
Sum	**12**	**679**	**5**	**25**	**116**
		P		**Mx**	**My**

Soil bearing pressure:

Area at base = 2 × 2 = 4 m^2
Zx = 2 × 2^2 / 6 = 1.33 m^3 Zy = 2 × 2 ^{2} / 6 = 1.33 m3
P/A = 679 / 4 = 170 kN/m^2
Mx / Zx = 25 / 1.33 = 19 kN/m^2 My / Zy = 116 / 1.33 = 87 kN/m^2

Maximum GBP (gross bearing pressure) at corners:

Corner 1: 170 + 19 + 87 = 276 kN/m^2 < Safe gross bearing pressure; safe.
Corner 2: 170 − 19 + 87 = 238 kN/m^2
Corner 3: 170 − 19−87 = 64 kN/m^2 No uplift at the corner.
Corner 4: 170 + 19−87 = 102 kN/m^2

Average soil bearing pressure = (276 + 238 + 64 + 102) / 4 = 170 kN/m^2 < gross bearing pressure; safe.

Gross soil bearing pressure, GBP = net SBP + γsoil × h
= 250 + 16 × 2.1 = 285 kN/m^2
Self-weight of the footing slab = ~0.45 × 25 = 15 kN/m^2
Weight of backfill above the footing slab = 1.65 × 16 = 26 kN/m^2
Net upward pressure = 170−15−26 = 129 kN/m^2 (for strength design of the slab.)

Stability check

Uplift – no uplift; so overturning check is not required.
Sliding –sliding force = $\sqrt{(12^2 + 5^2)}$ = 13 kN

Sliding-resistant capacity = μ × P = 0.33 × 679 = 224 kN
Factor of safety = 224 / 13 = 17 > 1.5; safe.

[For granular soil, the coefficient of friction μ is equal to tan ϕ, ϕ being the angle of friction of soil. The sliding stability on cohesive soil, the adhesion between the base slab and the soil is assumed to be equal to the cohesive strength of soil (c) multiplied by footing area.]

5.3.2.2 Strength design of a footing slab

Along the Y axis:

Critical shear plane from the face of the pedestal = X

X = $d_{effective}$ = (450−16−0.5 × 12) / 1000 = 0.369 m

Projection of a footing slab outside the pedestal,

Y = (2−0.45) / 2 = 0.775 m

Y − X = 0.775−0.369 = 0.406 m

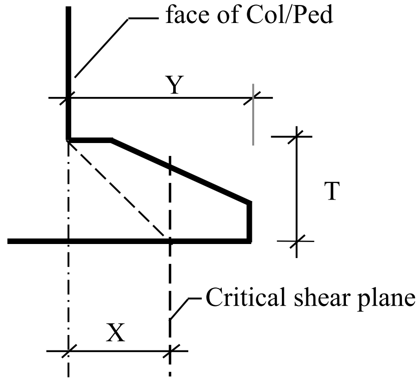

FIGURE 5.37 Part section at Y-Y

Shear area at the critical section:

See the plan view of Figure 5.56.

Width of the slab at the bottom = 2.0 m

Width of the slab at the top = *Lx*

0.5(L-l) = 0.55 m *0.5(B−b) = 0.625 m* *X − e = 0.22 m*

Lx = 0.9 + 2 × (0.219 × 0.55 / 0.625) = 1.285 m

Depth at the critical section = Tx = 0.45 × (0.625 − 0.219) / 0.625 = 0.292 m

Shear area at the critical section, Ay = 0.5 × (1.285 + 2) × 0.292 = 0.480 m^2

Net upward soil pressure = 129 kN/m^2

Moment = (2 × 0.775 × 129) × (0.775 / 2) = 78 kNm for full width
and 78 / 2.2 = 39 kNm per meter wide.

Shear = 2 × 0.406 × 129 = 105 kN for full width

Along the X axis:

Y = (2 − 0.6) / 2 = 0.7 m X = d = 0.369 m

Y − X = 0.331 m

Width of the slab = 2 m

Width of the slab at the top = *Bx*

0.5(B-b) = 0.63 m *0.5(L-l) = 0.55 m* *X − e = 0.22 m*

Bx = 0.75 + 2 × (0.219 × 0.625 / 0.55) = 1.248 m

Depth at the critical section = Tx = 0.45 × (0.55 − 0.219) / 0.55 = 0.271 m

Shear area at the critical section, Ax = 0.5 × (1.248 + 2) × 0.271 = 0.440 m^2

Net upward soil pressure = 129 kN/m^2

Moment = (2 × 0.7 × 129) × (0.7 / 2) = 63.21 kNm for full width and
63.21 / 2 = 32 kN/m meter width

Shear = = 2 × 0.331 × 129= 85 kN.

Provide reinforcement bars:

At the bottom layer

Along the X axis – long bars: 12 mm diameter @ 250 mm c/c (Ast = 453 mm^2 /m)

Along the Z axis – transverse bars: 12 mm diameter @ 250 mm c/c (Ast = 453 mm^2 /m)

Check strength design in conformity with the IS Code – IS: 456 and SP 16

Along the Y axis:

Moment at the face of the column = 39 kN/m load factor = 1.2
d prov = davg = [0.5 × (2 + 0.9) × 0.45 / 2] × 1000 = 326 mm
$Mu / bd^2 = 1.2 \times 39 \times 1000000 / (1000 \times 326^2) = 0.44$
pt = 0.127 % for M25 (from SP 16 Table 3)
$Ast_{reqd.} = 0.127 \times 1000 \times 326 / 100 = 414\ mm^2/m$ width
$Ast_{min} = 391\ mm^2/m$ width (0.12 %)
$Ast_{provided} = 453\ mm^2/m$ width; > Ast_{reqd}; safe.

Shear:

$pt_{provided} = 453 \times 100 / (1000 \times 326) = 0.14$ %
$\tau c = 0.29$ MPa for pt provided; (IS 456:2000 Table 19; SP 16 Table 61)
$\tau c.Ay = 0.29 \times 0.48 \times 10^6 / 1000 = 139$ kN where Ay = shear area
V = 105 kN Vu = 1.2 × 105 = 126 kN < τc.A; safe, shear reinforcement not required.

Along the X axis:

Moment at the face of the column = 32 kN/m d avg = 309 mm Load factor = 1.2
$Mu / bd^2 = 1.2 \times 32 \times 1000000 / (1000 \times 309^2) = 0.40$
pt = 0.113 % for M25 (from SP 16 Table 3)
$Ast_{reqd.} = 349\ mm^2/m$ width
$Ast_{min} = 371\ mm^2/m$ width
$Ast_{provided} = 453\ mm^2/m$ width; > Ast_{reqd}; safe.

Shear:

$pt_{provided} = 453 \times 100 / (1000 \times 309) = 0.15$ %
$\tau c = 0.29$ MPa for pt provided; (IS 456:2000 Table 19; SP 16 Table 61)
$\tau c.Ax = 0.29 \times 0.44 \times 10^6 / 1000 = 128$ kN
V = 85 kN Vu = 1.2 × 85 = 102 kN < τc.A; safe, shear reinforcement not required.

Shrinkage reinforcement at the top layer – both ways:

Provide 12 mm diameter @ 400 mm c/c (Ast = 283 mm^2 /m) > 185 mm^2 /m; okay.
Half of minimum reinforcement = 185 mm^2 /m

Design of the pedestal will be the same as the design of the column (IS 456 and SP16).

As per the ACI Code – ACI 318–19 in US Customary Unit

Refer to Figure 5.55 Stress-strain distribution diagram in the slab above

Design parameters:

fc′ = 3000 psi fy = 60000 psi Es = 29000000 psi
One-way slab bw = 12 inch h = 18 inch (max)

Cover, $d' = 3$ inch $d_{average} = 13$ inch

Design load (factored) ACI 318–19 (Table 5.3.1)
Mu Long = 11 kip-ft / ft
Mu Tranv = 9 kip-ft / ft
Vu = 4 kip /ft

Main bars at the bottom layer:

Along the Y axis bottom layer – bar no. # 4 @ 8 inch c/c 0.30 in^2 / ft
Top layer – bar no # 4 @ 10 inch c/c 0.24 in^2 / ft
Total = 0.54 in^2 / ft > minimum reinforcement

Along the X axis bottom layer – bar no. # 4 @ 8 inch c/c 0.30 in^2 / ft
Top layer – bar no. # 4 @ 10 inch c/c 0.24 in^2 / ft
Total = 0.54 in^2 / ft > minimum reinforcement

Minimum reinforcement:
As min = 0.0018 Ag = 0.0018 × 12 × 13 = 0.28 in^2 / ft ACI 318–19 (7.6.1.1)

Flexural reinforcement as a singly reinforced slab

Along the Y axis:

Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu =11 kip-ft nominal strength = Mn; ϕ Mn > = Mu ACI 318–19 (9.5.1.1)
From Figure 5.55 – stress and strain diagram for the slab above,

C = T 0.85 fc′ b a = As fy effective width b = bw = 12 inch
a = As fy / 0.85 fc′ b = 60000 / (0.85 × 3000 × 12) × As = 1.961 As
Ast provided = 0.30 in^2 /ft
So, a = 1.961 × 0.3 = 0.588 inch
ϕ Mn = ϕ As fy (d – a/2)
ϕ Mn = 0.9 × 0.3 × 60000 × (13 – 0.5 × 0.588) = 203 kip-inch = 17 kip-ft > Mu; safe.

Along the X axis:

Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 9 kip-ft nominal strength = Mn; ϕ Mn > = Mu ACI 318–19 (9.5.1.1)
From Figure 5.55 – stress and strain diagrams for the slab above,

C = T0.85 fc′ b a = As fy effective width b = bw = 12 inch

a = As fy / 0.85 fc′ b = 60000 / (0.85 × 3000 × 12) × As = 1.961 As
Ast provided = 0.3 in^2 /ft
So, a = 1.961 × 0.3 = 0.588 inch
ϕ Mn = ϕ As fy (d – a/2)
ϕ Mn = 0.9 × 0.3 × 60000 × (13 – 0.5 × 0.588) = 203 kip-inch = 17 kip-ft > Mu; safe.

Shear:

ϕ Vn > = Vu ACI 318–19 (9.5.1.1)
Vn = Vc + Vs ACI 318–19 (9.5.3.1; 22.5.1.1)
Vc = 2 $\sqrt{fc'}$.bw.d ACI 318–19 (Table 22.5.1.1)

Required nominal strength =Vn
Strength reduction factor, $\phi = 0.75$ for shear ACI 318–19 (Table 21.2.1)
Vu = 4 kip

Shear capacity
ϕ Vc = $\phi \times 2\ \lambda \sqrt{fc'}$. bw. d = $0.75 \times 2 \times 0.94 \times \sqrt{(3000)} \times 12 \times 13 / 1000 = 12$ kips
$\lambda = \sqrt{[2/(1 + 0.1d)]} = 0.94 < 1$ ACI 318–19 (Table 22.5.5.1a)
ϕ Vc > Vu; shear reinforcement is not necessary. ACI 318–19 (22.5.5.1.3)

Maximum allowable shear in this C/S of the slab, Vu max = 51 kips ACI 318–19 (22.5.1.2)
Vu max <= ϕ (Vc + 8 $\sqrt{fc'}$. bw.d)
Vu < Vu max

Re-bars at Top (along Y and X axes) ACI 318–19 (24.4.3.2)
Shrinkage reinforcement (0.0018Ag = 0.28 inch^2) may be distributed in two layers. There are tension reinforcement bars in the bottom layer, so provide half of the minimum re-bars (0.0009Ag = 0.14 inch^2) at the top.
Provided re-bars: Bar # 4 @ 10 inch centers (0.24 inch^2) in both directions at the top layer.
Hence, safe.
Design of the pedestal shall be the same as the design of the column.

As per the ACI Code – ACI 318–19 SI Unit

Refer to Figure 5.55.

Design parameters:

fc′ = 20 MPa fy = 415 MPa Es = 200000 MPa
One-way slab bw = 1000 mm h = 450 mm
Cover, d′ = 75 mm d average = 326 mm

Design load (factored) ACI 318–19 (Table 5.3.1)
Mu Long = 47 kNm
Mu Tranv = 38 kNm
Vu = 63 kN

Main bars at the bottom layer:

Along the Y axis	bottom layer: 12 mm diameter@ 200 mm c/c	(Ast = 566 mm^2 /m width)
	Top layer: 12 mm diameter@ 250 mm c/c	(Ast = 453 mm^2 /m width)
	Total = 1019 mm^2 /m width > minimum reinforcement	
Along the X axis	bottom layer: 12 mm diameter@ 200 mm c/c	(Ast = 566 mm^2 /m width)
	Top layer: 12 mm diameter@ 250 mm c/c	(Ast = 453 mm^2 /m width)
	Total = 1019 mm^2 /m width > minimum reinforcement	

Minimum reinforcement:
As min = 0.0018 Ag = 0.0018 × 1000 × 450 = 810 mm^2/m ACI 318–19 (7.6.1.1)

Along the Y axis:

Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 47 kNm nominal strength = Mn; ϕ Mn > = Mu ACI 318–19 (9.5.1.1)
From Figure 5.55 – stress and strain diagram for the slab above,

C = T0.85 fc′ bw a = As fy effective width b = bw = 1000 mm
a = As fy / 0.85 fc′ b = 566 × 415 / (0.85 × 20 × 1000) = 13.8 mm

ϕ Mn = ϕ As fy (d – a/2)
ϕ Mn = 0.9 × 566 × 415 × (326 – 0.5 × 13.8) = 67 kNm = > Mu; safe.

Along the X axis:

Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 38 kNm nominal strength = Mn; ϕ Mn > = Mu ACI 318–19 (9.5.1.1)
From Figure 5.55 – stress and strain diagram for the slab above,

C = T0.85 fc′ bw a = As fy effective width b = bw = 1000 mm

a = As fy / 0.85 fc′ b = 566 × 415 / (0.85 × 20 × 1000) = 13.8 mm

ϕ Mn = ϕ As fy (d – a/2)
ϕ Mn = 0.9 × 566 × 415 × (326 – 0.5 × 13.82) = 67 kNm = > Mu; Safe.

Shear:

Strength reduction factor, $\phi = 0.75$ for shear ACI 318–19 (Table 21.2.1)

Vu = 63 kN required nominal strength, V_n = Vu / ϕ = 84 kN

Vc = 0.17 λ $\sqrt{fc'}$. bw. d = 0.17 × 0.93 × $\sqrt{}$ 20 × 1000 × 326 / 1000 = 231kN
$\lambda = \sqrt{[2 / (1 + 0.004d)]} = 0.93 < 1$ ACI 318–19 (Table 22.5.5.1a)
Vc > Vu; shear reinforcement is not necessary. ACI 318–19 (22.5.5.1.3)

Maximum allowable shear in this C/S of the slab, Vu max = 722 kN ACI 318–19 (22.5.1.2)
Vu max <= ϕ (Vc + 0.66 $\sqrt{fc'}$. bw.d)
Vu < Vu max

Re-bars at top (along Y and X axes) ACI 318–19 (24.4.3.2)
Shrinkage reinforcement (0.0018Ag = 810 mm^2) may be distributed in two layers. There are tension reinforcement bars in the bottom layer, so provide half the minimum re-bars (0.0009Ag = 405 mm^2) at the top.
Provided re-bars: 12 mm diameter @ 250 mm C/C (Ast = 453 mm^2) in both directions at the top layer.
Hence, safe.
Design of the pedestal shall be the same as the design of the column.

5.4 DESIGN OF COMBINED FOOTING

5.4.1 Sketch

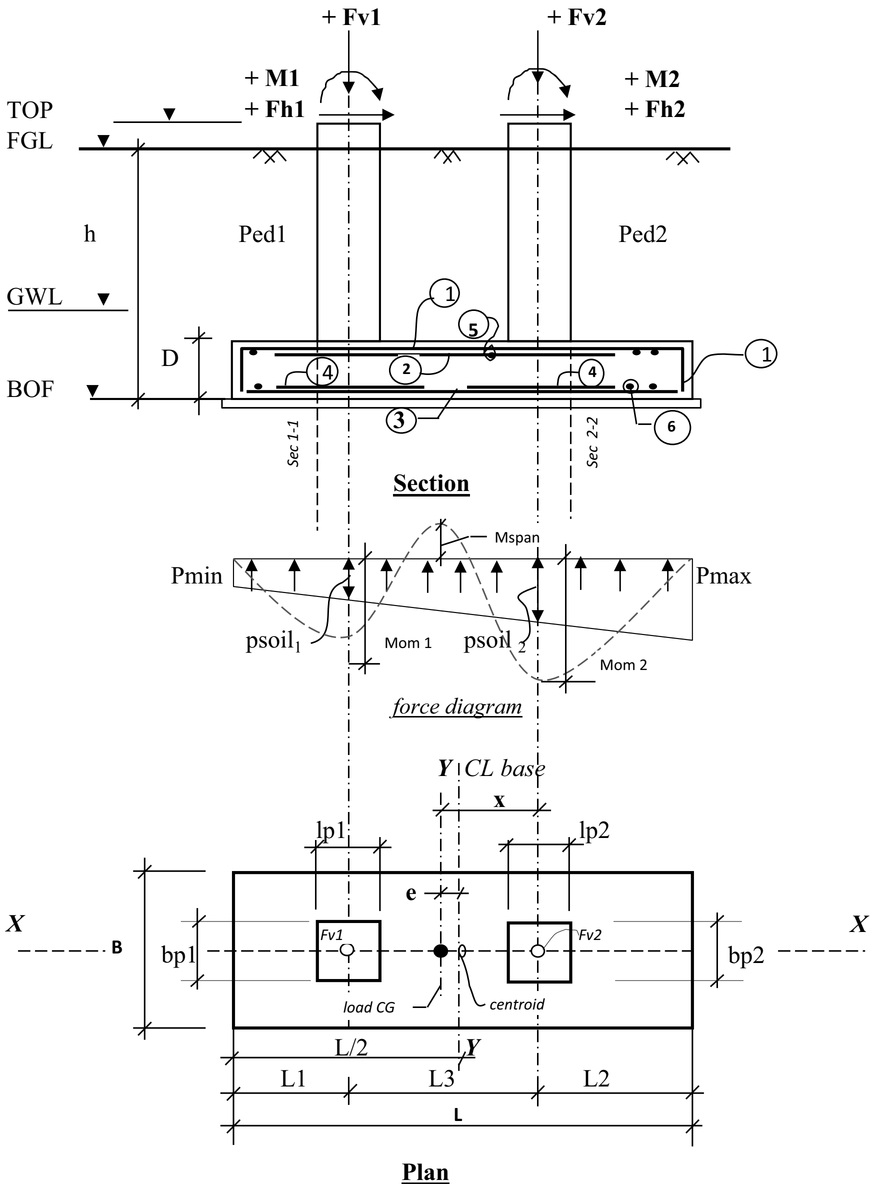

FIGURE 5.38 Combined footing

Design parameters

Finish-grade level (RL*) FGL = 243 M   Top of the pedestal (RL)   TOP = 243.3 M
Groundwater level (RL) GWL= 240 M   Bottom of the foundation (RL) BOF = 239.3 M
[*RL denotes the plant reference level]
Depth of the foundation below FGL, h = 3.7 m
Net safe bearing pressure, SBP = 200 kN/m^2
Gross bearing pressure, p_{gross} = (Net SBP + γsoil × h) = 267 kN/m^2
Surcharge, γ_{sur} = 5 kN/m^2 Unit weight of backfill soil, γ_{fill} = 18 kN/m^3
Unit weight of concrete, γ_{conc} = 25 kN/m^3
Coefficient of sliding friction in soil, μ_{soil} = 0.3
Grade of concrete = M25   fck = 25 MPa (cube strength)
Reinforcement steel = Fe415   fst = 415 MPa
ϕ diameter = 25 mm   concrete cover = 50 mm

Pedestal sizes:	*Foundation sizing:*	
lp1 = 0.6 m	L = 6.5 m	L1 = 1.25 m
bp1 = 0.6 m	B = 2.5 m	L2 = 1.25 m
lp2 = 0.6 m	D = 0.6 m	L3 = 4 m
bp2 = 0.6 m		

5.4.2 Reinforcement steel

TABLE 5.37
Reinforcement bar:

Bar mkd.	Description	Area provided mm^2 /m		Location
Along the X axis				
1	16 mm diameter @ 400 C/C	*503*	1006	Top layer
2	16 mm diameter @ 400 C/C	*503*		
3	16 mm diameter @ 400 C/C	*503*	1006	Bottom layer
4	16 mm diameter @ 400 C/C	*503*		
Along the Y axis				
5	16 mm diameter @ 250 C/C	805	1609	Distribution bar
6	16 mm diameter @ 250 C/C	805		

5.4.3 Foundation load

TABLE 5.38
Foundation load (unfactored)

Load cases	Description	Left side pedestal Fv1(*) kN	Fh1 kN	M1 kNm	Right side pedestal Fv2 kN	Fh2 kN	M2 kNm	Load factor Lf
1	DL + LL	**380**	**9.5**	**0**	**380**	**9.5**	**0**	1
2	DL+WL (L-R) +LL	**–74**	**38.5**	**0**	**833**	**38.5**	**0**	1
3	DL+WL (L to R)	**–200**	**37**	**0**	**700**	**37**	**0**	1
4	DL+%WL	**–87**	**18.5**	**0**	**367**	**18.5**	**0**	1
5	DL+LL+%WL	**39**	**20**	**0**	**493**	**20**	**0**	1

(*) Negative sign means uplift / tension

5.4.4 Soil bearing pressure and stability check

TABLE 5.39
Soil bearing pressure and stability check

	$M_{overturn}$	Soil pressure				Factor of safety against	
	M_0	P_{max}	P_{min}	$p_{soil}1$	$p_{soil}2$		
Load cases	**kNm**	**kN/m²**	**kN/m²**	**kN/m²**	**kN/m²**	**Overturn**	**Slide**
1	76	*121*	*112*	*114*	*119*	*81*	*30*
2	2122	*237*	*–4*	*42*	*191*	*2.0*	*7*
3	2096	*220*	*–18*	*27*	*174*	*1.7*	*7*
4	1056	*147*	*27*	*50*	*124*	*3.5*	*11*
5	1068	*163*	*42*	*65*	*140*	*4.2*	*13*
Observation		Safe	Safe	Safe		Safe	Safe

Note:
Allowable gross bearing pressure = 267 kN/m²
Factor of safety against overturning > 1.5
Factor of safety against sliding >= 1.5
Shear F = maximum of F1 and F2

5.4.5 Moment and shear forces

TABLE 5.40
Foundation moments and shear for strength design

	Moment – along the X axis			Moment – along the Y axis		Shear – along the X axis			
Load case	Mom1 kNm	Mom2 kNm	Mspan kNm	MTran1 kNm	MTran2 kNm	F kN	SF1 kN	SF2 kN	Load description
1	57	57	154	128	128	105	99	105	DL + LL
2	57	57	154	2.0	270	187	17	187	DL+WL (L–R)+LL
3	39	39	105	1.7	228	172	4	172	DL+WL L to R
4	24	24	64	3.5	124	119	34	119	DL+%WL
5	41.1	41	111	4.2	164	133	47	133	DL+LL+%WL

The cell values shown in tables are obtained by using visual basic application (VBA) functions in Excel. Calculation backup details are presented below for Load case 1.

5.4.5.1 Breakup calculation

Load Case 1:

Function Mo(FGL, BOF, Fv1, Fv2, Fh1, Fh2, M1, M2, L, L2, L3, lp1, bp1, lp2, bp2, D, TOP, GWL)

*Ped1 = lp1 * bp1 * ((TOP – BOF – D) * 25 – (GWL – BOF – D) * 10)*
*Ped2 = lp2 * bp2 * ((TOP – BOF – D) * 25 – (GWL – BOF – D) * 10)*
*x = (Fv1 + Ped1) * L3 / (Fv1 + Fv2 + Ped1 + Ped2)*
*e = 0.5 * L – (L2 + x)*
h = TOP – BOF
*Mo = (Fv1 + Ped1 + Fv2 + Ped2) * e + M1 + M2 + (Fh1 + Fh2) * h*

End function

Ped1 = 0.6 × 0.6 × ((243.3 − 239.3 − 0.6) × 25 − (240 − 239.3 − 0.6) × 10) = 30.24 kN
Ped 2 = 0.6 × 0.6 × ((243.3 − 239.3 − 0.6) × 25 − (240 − 239.3 − 0.6) × 10) = 30.24 kN
x = (380 + 30.24 + 380 + 30.24) × 4 / (380 + 380 + 30.24 + 30.24) = 2 m distance from CL Ped 2
e = 0.5 × 6.5 – (1.25 + 2) = 0
h = 243.3–239.3 = 4 m
M_0 = (380 + 30.24 + 380 + 30.24) × (0) + 0 + 0 + (9.5 + 9.5) x 4 = 76 kNm

Pmax = maximum pressure on soil

Function Pmax(FGL, GWL, BOF, gfill, gconc, Surcharge, L, B, D, Fv1, Fv2, Mo, lp1, bp1, lp2, bp2, TOP)

*Ped1 = lp1 * bp1 * ((TOP – BOF – D) * 25 – (GWL – BOF – D) * 10)*
*Ped2 = lp2 * bp2 * ((TOP – BOF – D) * 25 – (GWL – BOF – D) * 10)*
*Area C = L * B*
*Area S = L * B – lp1 * bp1 – lp2 * bp2*
*Wbase = AreaC * D * gconc + AreaS * (FGL – BOF – D) * gfill + (Areas * Surcharge) – (AreaC * (GWL – BOF) * 10)*
*Pmax = (1 / (B * L)) * ((Fv1 + Ped1 + Fv2 + Ped2 + Wbase) + (Mo * 6) / L)*

End function

Ped1 = 30.24 kN
Ped2 = 30.24 kN
Area C= area of the footing slab in contact with soil = L x B
Area C= 6.5 × 2.5 = 16.25 m^2
Area S = area of backfill earth above the footing slab = L × B – lp1 × bp1 – lp2 × bp2
Area S = 6.5 × 2.5 – 0.6 × 0.6 – 0.6 × 0.6 = 15.53 m^2
Wbase = self-weight of footing slab, weight of backfill earth and surcharge on ground
= 16.25 × 0.6 × 25 + 15.53 × (243 − 239.3 − 0.6) × 18 + 15.53 × 5 − (16.25 × (240 − 239.3) × 10)
= 1074 kN
Pmax = (1 / (2.5 × 6.5)) × ((380 + 30.24 + 380 + 30.24 + 1074) + (76 × 6) / 6.5) = 121 kN/m^2
= 121 kN/m^2

Function Pmin(FGL, GWL, BOF, gfill, gconc, Surcharge, L, B, D, Fv1, Fv2, Mo, lp1, bp1, lp2, bp2, TOP)

*Ped1 = lp1 * bp1 * ((TOP – BOF – D) * 25 – (GWL – BOF – D) * 10)*
*Ped2 = lp2 * bp2 * ((TOP – BOF – D) * 25 – (GWL – BOF – D) * 10)*
*Area C = L * B*
*Areas = L * B – lp1 * bp1 – lp2 * bp2*
*Wbase = AreaC * D * gconc + Areas * (FGL – BOF – D) * gfill + (Areas * Surcharge) – (AreaC * (GWL – BOF) * 10)*
*Pmin = (1 / (B * L)) * ((Fv1 + Ped1 + Fv2 + Ped2 + Wbase) – (Mo * 6) / L)*

End function

Pmin = minimum pressure on soil

Pmin = (1 / (2.5 × 6.5)) × ((380 + 30.24 + 380 + 30.24 + 1074) − (76 × 6) / 6.5) = 112 kN/m^2

psoil1 = soil pressure below pedestal 1

Function psoil1 (L, L1, Pmax, Pmin)
*psoil1 = Pmin + (L1 / L) * (Pmax – Pmin)*
End function
psoil1 = 112 + (1.25 / 6.5) × (121 − 112)

psoil2 = soil pressure below pedestal 2

Function psoil2 (L, L1, L3, Pmax, Pmin)
*psoil2 = Pmin + ((L1 + L3) / L) * (Pmax – Pmin)*
End function

psoil2 = 112 + ((1.25 + 4) /6.5) × (121 − 112) = 119 kN/m^2

FOSovr = factor of safety against overturning
Function FOSovr (Fv1, Fv2, Mo, FGL, GWL, BOF, L, L2, L3, B, D, gfill, gconc, Surcharge, lp1, bp1, lp2, bp2, TOP)

*Ped1 = lp1 * bp1 * ((TOP – BOF – D) * 25 – (GWL – BOF – D) * 10)*
*Ped2 = lp2 * bp2 * ((TOP – BOF – D) * 25 – (GWL – BOF – D) * 10)*
*AreaC = L * B*
*Areas = L * B – lp1 * bp1 – lp2 * bp2*
*Wbase = AreaC * D * gconc + Areas * (FGL – BOF – D) * gfill + (Areas * Surcharge) – (AreaC * (GWL – BOF) * 10)*
*FOSovr = ((Fv1 + Ped1) * (L2 + L3) + (Fv2 + Ped2) * L2 + Wbase * 0.5 * L) / Mo*

End function

Ped 1 = 30.24 kN Ped2 = 30.24 kN Wbase = 1074 kN
M_0 = 76 kNm
FOSovr = ((380 + 30.24) × (1.25 + 4) + (380 + 30.24) × 1.25 + 1074 × 0.5 × 6.5) / 76
= 81

Function FOSslide (CoeffFrixn, Fv1, Fv2, Fh1, Fh2, FGL, GWL, BOF, L, B, D, gfill, gconc, Surcharge, lp1, bp1, lp2, bp2, TOP)

*Ped1 = lp1 * bp1 * ((TOP – BOF – D) * 25 – (GWL – BOF – D) * 10)*
*Ped2 = lp2 * bp2 * ((TOP – BOF – D) * 25 – (GWL – BOF – D) * 10)*
*AreaC = L * B*
*Areas = L * B – lp1 * bp1 – lp2 * bp2*
*Wbase = AreaC * D * gconc + Areas * (FGL – BOF – D) * gfill + (Areas * Surcharge) – (AreaC * (GWL – BOF) * 10)*
*FOSslide = (Fv1 + Ped1 + Fv2 + Ped2 + Wbase) * CoeffFrixn / (Fh1 + Fh2)*
FOSslide = (380 + 30.24 + 380 + 30.24 + 1074) × 0.3 / (9.5 + 9.5) = 30

Along the X axis

Moment at Section 1–1, Mom 1

Function Moment1 (FGL, GWL, BOF, L, L1, B, D, gfill, Surcharge, Pmin, Pmax, lp1, lp2, bp1, bp2, gconc)

*AreaC = L * B*
*Areas = L * B – lp1 * bp1 – lp2 * bp2*
*Wbase = AreaC * D * gconc + Areas * (FGL – BOF – D) * gfill + (Areas * Surcharge) – (AreaC * (GWL – BOF) * 10)*
pbase = Wbase / AreaC
*Moment1 = B * (0.5 * (Pmin + Pmax) – pbase) * (L1 – 0.5 * lp1) ^ 2 / 2*

End function

AreaC = 16.25 m^2 AreaS = 15.53 m^2 Wbase = 1074 kN
pbase = 1074 / 16.25 = 66.09 kN/m^2

Moment 1 = $2.5 \times (0.5 \times (112 + 121) - 66.09) \times (1.25 - 0.5 \times 0.6)^2 / 2 = 56.87$ kNm

Moment at Section 2–2, Mom 2

Function Moment2 (FGL, GWL, BOF, L, L2, B, D, gfill, Surcharge, Pmax, Pmin, lp1, lp2, bp1, bp2, gconc)

*AreaC = L * B*
*Areas = L * B – lp1 * bp1 – lp2 * bp2*
*Wbase = AreaC * D * gconc + Areas * (FGL – BOF – D) * gfill + (Areas * Surcharge) – (AreaC * (GWL – BOF) * 10)*
pbase = Wbase / AreaC
*Moment2 = B * (0.5 * (Pmin + Pmax) – pbase) * (L2 – 0.5 * lp2) ^ 2 / 2*

End function

Moment 2 = $2.5 \times (0.5 \times (112 + 121) - 66.09) \times (1.25 - 0.5 \times 0.6)^2 / 2 = 56.87$ kNm

Span moment, Mspan (=Moment X):

Function Mspan(FGL, GWL, BOF, L, L1, L2, L3, B, D, gfill, Surcharge, Pmin, Pmax, lp1, lp2, bp1, bp2, gconc)

*AreaC = L * B*
*Areas = L * B – lp1 * bp1 – lp2 * bp2*
*Wbase = AreaC * D * gconc + Areas * (FGL – BOF – D) * gfill + (Areas * Surcharge) – (AreaC * (GWL – BOF) * 10)*
pbase = Wbase / AreaC
*c = 0.5 * (L1 + L2)*
*q = 0.5 * (Pmax + Pmin) – pbase*
*Mspan = B * (q / 8) * (L3 ^ 2–4 * c ^ 2)*

End function

Pmax = 121 kN/m^2 Pmin = 112 kN/m^2 pbase = 66.09 kN/m^2

c = 0.5 × (1.25 + 1.25) = 1.25 m L3 = 4 m
q = 0.5 × (121 + 112) – 66.09 = 50.41 kN/m^2
M Span = 2.5 × (50.41 / 8 x (4^2 – 4 x 1.25^2) = 154 kNm.

Shear force, F

at Section 1–1

Function SF1(B, psoil1, Pmin, L1, lp1, D)
*pavg1 = 0.5 * (psoil1 + Pmin)*
*SF1 = B * pavg1 * (L1 – 0.5 * lp1 – D)*
End function
pavg1 = 0.5 × (114 + 112) = 113 kN/m^2
SF1 = 2.5 × 113 × (1.25 – 0.5 × 0.6 – 0.6 = 98.88 kN

at Section 2–2

Function SF2(B, psoil2, Pmax, L2, lp2, D)
*pavg2 = 0.5 * (psoil2 + Pmax)*
*SF2 = B * pavg2 * (L2 – 0.5 * lp2 – D)*
End function
pavg2 = 0.5 × (119 + 121) = 120 kN/m^2
SF2 = 2.5 × 120 × (1.25 – 0.5 × 0.6 – 0.6) = 105 kN/m^2

Along the Y axis

Transverse moment

Function MomTransv1(B, Fv1, lp1, bp1, D, TOP, BOF, GWL)

*Ped1 = lp1 * bp1 * ((TOP – BOF – D) * 25 – (GWL – BOF – D) * 10)*
*MomTransv1 = (1 / 8) * B * (Fv1 + Ped1)*

End function

Effective width = 2 L1 = 2 × 1.2 = 2.5 m
Mom Transv1 = (1/8) × 2.5 × (380 + 30.24) = 128.2 kNm
Function MomTransv2(B, Fv2, lp2, bp2, D, TOP, BOF, GWL)

*Ped2 = lp2 * bp2 * ((TOP – BOF – D) * 25 – (GWL – BOF – D) * 10)*
*MomTransv2 = (1 / 8) * B * (Fv2 + Ped2)*

End function

Effective width = 2 L2 = 2 × 1.2 = 2.5 m
Mom Transv2 = (1/8) × 2.5 × (380 + 30.24) = 128.2 kNm

Shear force should be the same as along the X axis.

5.4.5.2 Strength design

~~5.4.5.2.1~~ As per the IS Code – IS: 456 and SP 16

The footing slab is designed to resist the design moment and shear, as stated in Table 5.40.

Along the X axis

a) *Forces at the pedestal face: (factored load; load factor, LF = 1.5/1.2)*

Bottom layer (bar 3 + 4)

M = Mom1/Mom2: DL + LL = 1.5 × 57 = 86 kNm LF = 1.5
DL + LL + WL = 1.2 × 57 = 68 kNm LF = 1.2

Design moment, Mu = 86 / 2.5 = 34 kN/m wide (width, B = 2.5 M)

Top layer (bar 1 + 2)

M = Mom1/Mom2: DL + LL = 1.5 × 0 = 0 kNm
DL + LL + WL = 1.2 × 24 = 29 kNm

Design moment, Mu = 29 / 2.5 = 12 kN/m wide (width, B = 2.5 M)

Shear, F: DL + LL = 1.5 × 105 = 158 kN LF = 1.5
DL + LL + WL = 1.2 × 187 = 224 kN LF = 1.2

Design shear, Vu = 224 / 2.5 = 90 kN/m wide (width, B = 2.5 M)

Flexural strength:

Bottom bar – 3 + 4

Mu = 34 kN/m d prov = 550 mm
$Mu/bd^2 = 34 \times 1000000 / (1000 \times 550^2) = 0.11$
pt = 0.084 % for M25 (from SP 16 Table 3)
$Ast_{reqd.} = 0.084 \times 1000 \times 550 / 100 = 462\ mm^2/m$ width
$Ast_{min} = 660\ mm^2/m$ width (0.12 %)
$Ast_{provided} = 1006\ mm^2/m$ width; > Ast_{reqd}; safe.

Top bar = 1 + 2

Mu = 12 kN/m d prov = 550 mm
$Mu/bd^2 = 12 \times 1000000 / (1000 \times 550^2) = 0.04$
pt = 0.084 % for M25 (from SP 16 Table 3)
$Ast_{reqd.} = 0.084 \times 1000 \times 550 / 100 = 462\ mm^2/m$ width
$Ast_{min} = 660\ mm^2/m$ width (0.12 %)
$Ast_{provided} = 1006\ mm^2/m$ width; > Ast_{reqd}; safe.

Shear strength:

$pt_{provided} = 1006 \times 100 / (1000 \times 550) = 0.18\ \%$

τc = 0.311 MPa for pt provided; (IS 456:2000 Table 19; SP 16 Table 61)
$\tau c.b.d$ = 0.311 × 1000 × 550 / 1000 = 171 kN
Vu = 90 kN < $\tau c.b.d$; safe, shear reinforcement not required

b) *Forces at middle span:* *(factored load; load factor = 1.5/1.2)*

$M = M_{span}$: DL + LL = 1.5 × 154 = 231 kNm
DL + LL + WL = 1.2 × 154 = 185 kNm

Design moment, Mu_{span} = 231 / 2.5 = 92 kN/m wide (width, B = 2.5 M)

Mu_{span} = 92 kN/m d prov = 550 mm
$Mu_{span}/bd^2 = 92 \times 1000000 / (1000 \times 550^2) = 0.3$
pt = 0.099 % for M25 (from SP 16 Table 3)
$Ast_{reqd.}$ = 0.099 × 1000 × 550 / 100 = 545 mm²/m width
Ast_{min} = 660 mm²/m width (0.12 %)
$Ast_{provided}$ = 1006 mm²/m width; (top bar 1 + 2) > Ast_{reqd}; safe.

Along the Y axis

a) *At the pedestal base:*

M = Mtranv1/Mtranv2: DL + LL = 1.5 × 128 = 192 kNm
DL + LL + WL = 1.2 × 270 = 324 kNm

Design moment, Mu_{tranv} = 324 / 2.5 = 130 kN/m wide

Shear, F: DL + LL = 1.5 × 89 = 134 kN
DL + LL + WL = 1.2 × 159 = 191 kN
Design shear, Vu = 191 / 2.5 = 76 kN/m wide (effective width = 2.5 M)

Flexural strength:

Mu = 130 kN/m d prov = 550 mm
$Mu/bd^2 = 130 \times 1000000 / (1000 \times 550^2) = 0.43$
pt = 0.127 % for M25 (from SP 16 Table 3)
$Ast_{reqd.}$ = 0.127 × 1000 × 550 / 100 = 699 mm²/m width
Ast_{min} = 660 mm²/m width (0.12 %)

Bar mark – 6

$Ast_{provided}$ = 805 mm²/m width; > Ast_{reqd}; safe.

Shear strength:
$pt_{provided}$ = 805 × 100 / (1000 × 550) = 0.15 %
τc = 0.29 MPa for pt provided; (IS 456:2000 Table 19; SP 16 Table 61)
$\tau c.b.d$ = 0.29 × 1000 × 550 / 1000 = 160 kN

Vu = 76 kN < τc.b.d; safe, shear reinforcement not required

Shrinkage reinforcement at the top layer – both ways:

Bar 5 or 6: provide 16 mm diameter @ 250 mm C/C (Ast = 805 mm^2)

[Half of minimum reinforcement = 330 mm^2]

Design of the pedestal shall be the same as the design of the column (IS 456 and SP16).

As per the ACI Code – ACI 318–19 (US Customary Unit)

Refer to Figure 5.55 Stress-strain distribution diagram in the slab above

Design parameters:

fc′ = 3000 psi fy = 60000 psi Es = 29000000 psi
One-way slab bw = 12 inch h = 24 inch (max)
Cover, d′ = 3 inch $d_{average}$ = 22 inch
Design load (factored) ACI 318–19 (Table 5.3.1)

Mx = 8 kip-ft / ft (34 kNm/m)

Mu Tranv = 21 kip-ft / ft (92 kNm/m)

Vu = 6 kip /ft (90 kN/m)

Main bars provided:

Bottom layer

Bar mkd. 3 - bar no. # 5 @ 14 inch c/c 0.27 in^2 / ft plus
Bar mkd. 4 - bar no. # 5 @ 14 inch c/c 0.27 in^2 / ft
Total = 0.54 in^2 / ft > minimum reinforcement

Top layer

Bar mkd. 1 - bar no. # 5 @ 14 inch c/c 0.27 in^2 / ft plus
Bar mkd. 2 - bar no. # 5 @ 14 inch c/c 0.27 in^2 / ft
Total = 0.54 in^2 / ft > minimum reinforcement

Minimum reinforcement:

As min = 0.0018 Ag = 0.0018 × 12 × 22 = 0.47 in^2 / ft ACI 318–19 (7.6.1.1)

Along the X direction:

Flexural reinforcement as a singly reinforced slab

Bottom layer:

Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 8 kip-ft nominal strength = Mn; ϕ Mn > = Mu ACI 318–19 (9.5.1.1)
From Figure 5.55 – stress and strain diagram for the slab above,

C = T 0.85 fc′ b a = As fy effective width b = bw = 12 inch
a = As fy / 0.85 fc′ b = 60000 / (0.85 × 3000 × 12) × As = 1.961 As
Ast provided = 0.54 in^2 /ft
So, a = 1.961 × 0.54 = 1.059 inch
ϕ Mn = ϕ As fy (d – a/2)
ϕ Mn = 0.9 × 0.54 × 60000 × (21.65 – 0.5 × 1.059) = 616 kip-inch = 51 kip-ft > Mu; safe.

Top layer:

Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 21 kip-ft nominal strength = Mn; ϕ Mn > = Mu ACI 318–19 (9.5.1.1)
From Figure 5.55 – stress and strain diagram for the slab above,

C = T 0.85 fc′ b a = As fy effective width b = bw = 12 inch

a = As fy / 0.85 fc′ b = 60000 / (0.85 × 3000 × 12) × As = 1.961 As
Ast provided = 0.54 in^2 /ft
So, a = 1.961 × 0.54 = 1.059 inch
ϕ Mn = ϕ As fy (d – a/2)
ϕ Mn = 0.9 × 0.54 × 60000 × (22 – 0.5 × 1.059) = 616 kip-inch = 51 kip-ft > Mu; safe.

Along the Y direction:

Moment transverse – Mtran 1 / Mtran2:
DL + LL =192 kNm (factored load)
DL + LL + WL = 324 kNm (factored load)
Design moment, Mtranv = 324 kNm = 324 / 2.5 = 130 kNm/meter wide = 29 kip-ft / ft

Bottom layer

Bar mkd. 6 - bar no. # 5 @ 10 inch c/c 0.37 in^2 / ft
Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 29 kip-ft nominal strength = Mn; ϕ Mn > = Mu ACI 318–19 (9.5.1.1)
From Figure 5.55 – stress and strain diagram for the slab above,

C = T 0.85 fc′ b a = As fy effective width b = bw = 12 inch

a = As fy / 0.85 fc′ b = 60000 / (0.85 × 3000 × 12) × As = 1.961 As
Ast provided = 0.37 in^2 /ft

So, a = 1.961 × 0.37 = 0.726 inch
ϕ Mn = ϕ As fy (d – a/2)
ϕ Mn = 0.9 × 0.37 × 60000 × (22 − 0.5 × 0.726) = 425 kip-inch = 35 kip-ft > Mu; safe.

Shear design:

ϕ Vn > = Vu ACI 318–19 (9.5.1.1)
Vn = Vc + Vs ACI 318–19 (9.5.3.1; 22.5.1.1)
Vc = 2 $\sqrt{fc'}$.bw.d ACI 318–19 (Table 22.5.1.1)
Required nominal strength =Vn
Strength reduction factor, ϕ = 0.75 for shear ACI 318–19 (Table 21.2.1)
Vu = 6 kip

Shear capacity of the concrete slab

ϕ Vc = ϕ × 2 λ $\sqrt{fc'}$. bw. d = 0.75 × 2 × 0.79 × $\sqrt{(3000)}$ × 12 × 22 /1000 = 17 kips
$\lambda = \sqrt{[2/(1 + 0.1d)]} = 0.79 < 1$ ACI 318–19 (Table 22.5.5.1a)
ϕ Vc > Vu; shear reinforcement is not necessary. ACI 318–19 (22.5.5.1.3)
Maximum allowable shear in this C/S of the slab, Vu max = 85 kips ACI 318–19 (22.5.1.2)
Vu max <= ϕ (Vc + 8 $\sqrt{fc'}$. bw.d)
Vu < Vu max

Re-bars at top (along Y and X axes) ACI 318–19 (24.4.3.2)
Shrinkage reinforcement (0.0018Ag = 0.47 inch^2) may be distributed in two layers. There are tension reinforcement bars at the bottom layer, so provide half of minimum re-bars (0.0009Ag = 0.23 inch^2) at the top.
Provided re-bars: Bar # 5 @ 10 inch centers (0.37 inch^2) in both directions at the top layer > 0.23 inch^2.
Hence, safe.
Design of the pedestal shall be the same as the design of the column.

As per the ACI Code – ACI 318–19 (SI Unit)

Refer to Figure 5.55 stress – strain distribution diagram in the slab above

Design parameters:

fc′ = 20 MPa fy = 415 MPa Es = 200000 MPa
One-way slab bw = 1000 mm h = 600 mm (max)
Cover, d′ = 50 mm $d_{average}$ = 550 mm
Design load (factored) ACI 318–19 (Table 5.3.1)
Mx = 34 kNm/m

Mu Tranv = 92 kNm/m

Vu = 90 kN/m

Main bars provided:

Bottom layer

Bar mkd. 3	– 16 mm @ 350 mm c/c	575 mm^2 / m	plus
Bar mkd. 4	– 16 mm @ 350 mm c/c	575 mm^2 / m	
	Total = 1150 mm^2 / m	> minimum reinforcement	

Top layer

Bar mkd. 1	– 16 mm @ 350 mm c/c	575 mm^2 / m	plus
Bar mkd. 2	– 16 mm @ 350 mm c/c	575 mm^2 / m	
	Total = 1150 mm^2 / m	> minimum reinforcement	

Minimum reinforcement:
As min = 0.0018 Ag = 0.0018 × 1000 × 600 = 1080 mm^2 ACI 318–19 (7.6.1.1)

Along the X direction:

Moment (Mom 1 / Mom 2) near face of pedestal, re-bars at the bottom layer **Bar 3+4**

Bottom layer:

Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 34 kNm nominal strength = Mn; ϕ Mn > = Mu ACI 318–19 (9.5.1.1)
From Figure 5.55 – stress and strain diagram for the slab above,

C = T 0.85 fc′ bw a = As fy

a = As fy / 0.85 fc′ bw = 575 × 415 / (0.85 × 20 × 1000) = 14.04 mm
ϕ Mn = ϕ As fy (d – a/2)
ϕ Mn = 0.9 × 1150 × 415 × (550 – 0.5 × 14.04) / 10^6 = 233 kNm > Mu; safe.

Top layer:

Mspan *Bar 1 + 2*

Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 92 kNm nominal strength = Mn; ϕ Mn > = Mu ACI 318–19 (9.5.1.1)
From Figure 5.55 – stress and strain diagram for the slab above,

C = T0.85 fc′ bw a = As fy

a = As fy / 0.85 fc′ bw = 575 × 415 / (0.85 × 20 × 1000) = 14.04 mm

ϕ Mn = ϕ As fy (d – a/2)

ϕ Mn = 0.9 × 1150 × 415 × (550 – 0.5 × 14.04) / 10^6 = 233 kNm > Mu; safe.

Along the Y direction:

Moment transverse – Mtran 1 / Mtran2:

DL + LL = 128 kNm (factored load)
DL + LL + WL = 270 kNm (factored load)
Design moment, Mtranv = 324 kNm = 324 / 2.5 = 130 kNm/meter-wide

Bottom layer

Bar mkd. 6 – 16 mm diameter @ 250 mm c/c (Ast = 805 mm^2 / m)
Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)
Mu = 130 kNm nominal strength = Mn; ϕ Mn > = Mu ACI 318–19 (9.5.1.1)
From Figure 5.55 – stress and strain diagram for the slab above,

C = T0.85 fc′ b a = As fy effective width b = bw = 1000 mm

a = As fy / 0.85 fc′ b = 805 × 415 / (0.85 × 20 × 1000) = 19.65 mm
ϕ Mn = ϕ As fy (d – a/2)
ϕ Mn = 0.9 × 805 × 415 × (550 – 0.5 × 19.65) /10^6 = 162 kNm > Mu; safe.

Shear design:

Strength reduction factor, ϕ = 0.75 for shear ACI 318–19 (Table 21.2.1)
Vu = 90 kN required nominal strength, V_n = Vu/ϕ = 120 kN

Shear capacity

Vc = 0.17 λ $\sqrt{fc'}$. bw. d = 0.17 × 0.79 × $\sqrt{20}$ × 1000 × 550 / 1000 = 331 kN
$\lambda = \sqrt{[2 / (1 + 0.004d)]} = 0.79 < 1$ ACI 318–19 (Table 22.5.5.1a)

ϕ Vc > Vu; shear reinforcement is not necessary. ACI 318–19 (22.5.5.1.3)

Maximum allowable shear in this C/S of the slab, Vu max = 1218 kN ACI 318–19 (22.5.1.2)
Vu max <= ϕ (Vc +.66$\sqrt{fc'}$. bw.d)
Vu < Vu max

Re-bars at Top (along Y and X axes) ACI 318–19 (24.4.3.2)

Shrinkage reinforcement (0.0018Ag = 1080 mm^2) may be distributed in two layers. There are tension reinforcement bars at the bottom layer, so provide half of minimum re-bars (0.0009Ag = 540 mm^2) at the top.
Provided re-bars: 16 mm diameter @ 250 mm centers (805 $inch^2$) in both directions at the top layer
> 540mm^2.
Hence, safe.
Design of the pedestal shall be the same as the design of the column.

5.5 DESIGN OF RAFT FOUNDATION

5.5.1 Turbine generator foundation raft of soil

5.5.1.1 Introduction

This workout example shows design of a 10MW turbine generator foundation resting on hard soil. The turbine foundation is an RCC framework consisting of a thick top table (slab) supported on six columns (C1 to C6). Turbine generator equipment is installed on the top table and held in position by anchor bolts. The foundation raft bears the load from the above six frame columns, weight of condenser and cooler units. The condenser and cooler units rest on the pedestal's cast on the base raft.

The base raft is designed as a large combined footing resting on hard soil. The strength design of the RCC raft is done in accordance with the IS code and ACI code.

5.5.1.2 Sketches

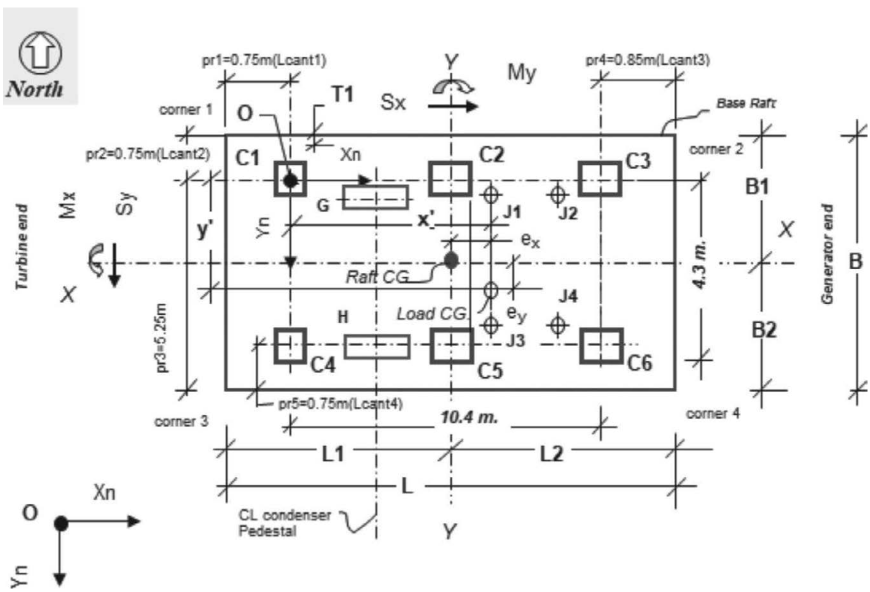

FIGURE 5.39 Plan view of the base raft

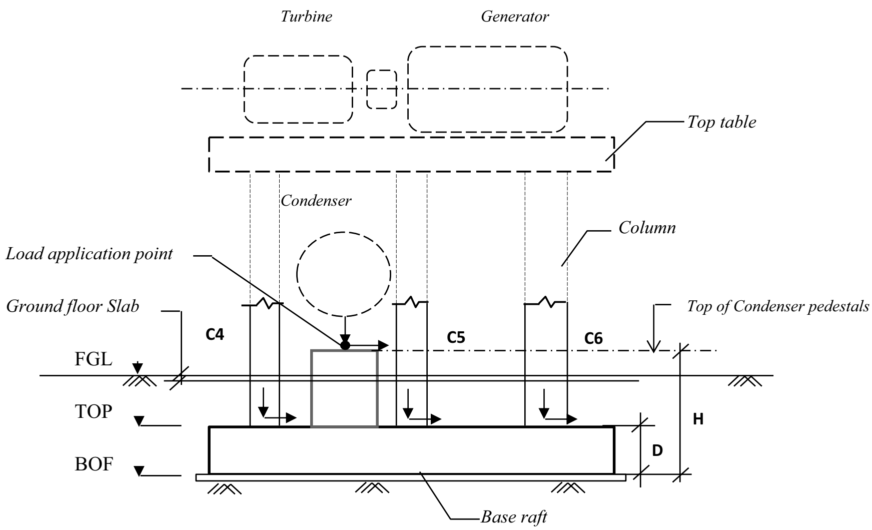

FIGURE 5.40 Elevation of turbine – generator foundation

[H = height of the load application point. D = thickness of the raft.]

Design parameters:

FGL	103.2 M	Finish-grade level [ground floor Elev. 0.0]
TOP	102.2 M	top of the foundation raft
BOF	101 M	bottom of foundation
SBP	350 kN/m^2	allowable soil bearing pressure at the founding level
fck	25 N/mm^2	M25 grade of foundation concrete
fst	415 N/mm^2	yield stress of reinforcement bars
γsoil	18 kN/m^3	unit weight of backfill earth
γconc	25 kN/m^3	unit weight of concrete

Raft sizing:

$L = 12$ m $\quad L_1 = 5.8$ m $\quad L_2 = 6.2$ m

$B = 6$ m $\quad B_1 = 3$ m $\quad B_2 = 3$ m

$D = 1.2$ m $\quad$ cover = 75 mm $\quad \phi = 25$ mm

deff = (1200−75−0.5 × 25) / 1000 = 1.11 m

Effective span of the base raft:

$Lx_{span} = 4.3$ m along Yn

$Ly_{span} = 5.35$ m along Xn

x = (Sum P × Xn +My + Sx × H) / Sum P m

x = (Sum P × Xn +My + Sx× H) / Sum P m

Resultant forces:

R = Sum P kN

$Mx' = R \times e_y$ kNm $\quad My' = R \times e_y$ kNm

$e_y = B \times 0.5 - pr_2 - y$ m $\quad e_x = L \times 0.5 - pr_1 - x$ m

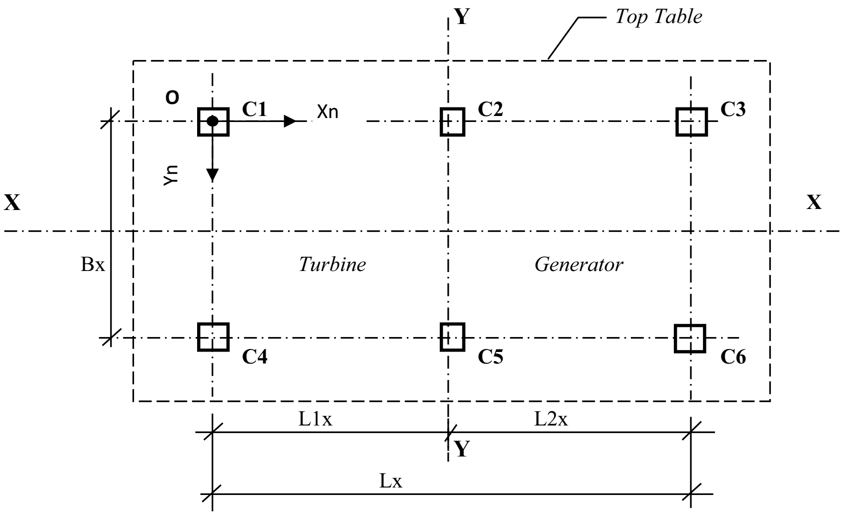

FIGURE 5.41 Column location plan

Bx = 4.3 m L1x = 5.05 m L2x = 5.35m Lx = 10.4m

TABLE 5.41
Column dimensions

Column	Length M	Width m
C1	1	1
C2	1.2	1
C3	1.2	1
C4	1	1
C5	1.2	1
C6	1.2	1

Reinforcement provided:

Along the X axis

Top 25 mm diameter bars @ 300 c/c Ast provided = 1637 mm^2
Bottom 25 mm diameter bars @ 300 c/c Ast provided = 1637 mm^2

Along the Y axis

Top 25 mm diameter bars @ 300 c/c Ast provided = 1637 mm^2
Bottom 25 mm diameter bars @ 300 c/c Ast provided = 1637 mm^2
Minimum shrinkage reinforcement = 0.0018 Ag = $0.0018 \times 1200 \times 1000$ = 2160 mm^2

Re-bars along the X axis= 1637 + 1637 = 3274 mm^2 > minimum reinforcement required.

Re-bars along the Y axis = 1637 + 1637 = 3274 mm^2 > minimum reinforcement required.

Design limitation

As per IS 2973 Part 3 and ACI 351.3 R-04

i) SBP should not exceed 80 % of the net safe bearing capacity of soil.

ii) Minimum reinforcement = 0.12% of gross area in each direction at the top and bottom.

Volumetric re-bars = 50 Kg/m3

Minimum spacing of reinforcement bars = 200 mm

Maximum spacing of reinforcement bars = 300 mm

iii) Eccentricity of the foundation load CG: Up to 3% of the base dimension, along which the center of gravity of load (weight of equipment and foundation) get displaced from the center of gravity of the base area in contact with soil. (Up to 5% – ACI 351.3 R-4).

iv) Minimum thickness = 0.07L 4/3; L is the average of two adjacent clear spans.
[1/5 of width (short side); 1/10 of length (long side) or 1/30 of the length plus 0.6 m as per ACI 351.3 R-04]

v) Design should be done in conformity with IS 456–1978 / ACI 318

Load on the base raft and soil bearing pressure:

Load Case 1: D+L+O+T (Dead load + Live load + Operating load + Torque)

Load Case 2: D+L+O+T + SCF (Dead load + Live load + Operating load + Torque + Short circuit force)

Load Case 3: D+L+O+T − SCF (Dead load + Live load + Operating load + Torque – Short circuit force)

Load Case 4: D+L+O+T+ FL(Dead load + Live load + Operating load + Torque + Failure load)

Load Case 5: D+L+O+T+SL (Dead load + Live load + Operating load + Torque + Seismic load)

Load Case 6: DL +LL+ O (Dead load + Live load + Operating load)

Load Case 7: DL +LL+ O (Dead load + Live load + Operating load)

Soil bearing pressure:

Resultant forces: R = Σ vertical load, Mx′ = Σ moment about the X axis, My′ = Σ moment about the Y axis

$p_1 = R / L.B - Mx'. 6 / L.B^2 - My'. 6 / B.L^2$

$p_2 = R / L.B - Mx'. 6 / L.B^2 + My'. 6 / B.L^2$

$p_3 = R / L.B + Mx'. 6 / L.B^2 - My'. 6 / B.L^2$

$p_4 = R / L.B + Mx'. 6 / L.B^2 + My'. 6 / B.L^2$

To find out CG of load for all the load cases:

TABLE 5.42
Load Case 1

D + L + O + T (Dead load + Live load + Operating load + Torque)

Item/ Load point	Magnitude (#1) P	Sx	My	Sy	Mx	Location Xn	Yn	Pedestal size Load Elev (RL)	length [yy]	width [xx]
	kN	kN	kNm	kN	kNm	m	m	M	m	m
1. C1 TG COL S-E	637		36		27	0	0	102.2	1	1
2. C2 TG COL SOUTH	1105		48		6	5.05	0	102.2	1	1.2
3. C3 TG COL S-W	771		48		34	10.4	0	102.2	1	1.2
4. C4 TG COL N-E	657		39		6	0	4.3	102.2	1	1
5. C5 TG COL NORTH	1051		48		39	5.05	4.3	102.2	1	1.2
6. C6 TG COL N-W	681		46		36	10.4	4.3	102.2	1	1.2
7. COND PED G	270			−75		2.343	0.425	104.3	0.7	2
8. COND PED H	270			75		2.343	4.425	104.3	0.7	2
9. GEN. COOLER J1	12					6.625	0.138	104.7	0.3	2
10. GEN.COOLER J2	12					8.825	0.138	104.7	0.3	2
11. GEN.COOLER J3	12					6.625	0.511	104.7	0.3	2
12. GEN.COOLER J4	12					8.825	0.511	104.7	0.3	2
13. GEN COOLER DUCT	594					13.08	1.765	102.2	3.3	4.15
14. MISC	0					0	0	0	0	0
15. BACKFILL	1296					5.2	2.15	102.2		
16. RAFT	2160					5.2	2.15	101		

Note: (#1) Load values P, Sx, My, Sy and Mx are column loads obtained from superstructure (top table and columns) frame analysis. These values include the self-weight of machine, top table and columns and operating loads. Condenser load (G.H) and generator cooler weight (J1 to J4) are directly resting on the base raft.

TABLE 5.43
CG of Load Case 1

D+L+O+T (Dead load + Live load + Operating load + Torque)

Item/ Load point	**Ped wt P1**	**Σ P = P + P1**	**ΣP * Xn**	**My+ Sx*H**	**x′**	**ΣP * Yn**	**Mx+ Sy*H**	**y′**	**Resultant forces R**	**Mx′**	**My′**
0	**kN**	**kN**	**kNm**	**kNm**	**m**	**kNm**	**kNm**	**m**	**kN**	**kNm**	**kNm**
1. C1 TG COL S-E	0.00	637	0	36	5.60	0	27	2.10	9841	1476	−3444
2. C2 TG COL SOUTH	0.00	1105	5580	48		0	6				
3. C3 TG COL S-W	0.00	771	8018	48		0	34		Soil bearing pressure		
4. C4 TG COL N-E	0.00	657	0	39		2825	6		p1	140	kN/m².
5. C5 TG COL NORTH	0.00	1051	5308	48		4519	39		p2	92	kN/m².
6. C6 TG COL N-W	0.00	681	7082	46		2928	36		p3	181	kN/m².
7. COND PED G	75.25	345	809	0		147	−251		p4	133	kN/m².
8. COND PED H	75.25	345	809	0	e_x	1528	251.3	e_y	pavg	137	kN/m².
9. GEN.COOLER J1	37.50	50	328	0	(–) **0.35**	7	0	**0.15**		**< SBP; safe**	
10. GEN.COOLER J2	37.50	50	437	0	m	7	0	m			
11. GEN.COOLER J3	37.50	50	328	0		25	0				
12. GEN.COOLER J4	37.50	50	437	0		25	0				
13. GEN COOLER DUCT	0.00	594	7769	0		1049	0				
14. MISC	0.00	0	0	0		0	0				
15. BACKFILL	0.00	1296	6739	0		2786	0				
16. RAFT	0.00	2160	11232	0		4644	0				
Sum		9841	54876	265		20491	148				

Note: Pedestal weight = length (yy) × wide (xx) × Height (Top Elev – TOC) x γconc

$x' = (54876 + 265) / 9841 = 5.6$ m $e_x = 0.5 \times 12 – 0.75 – 5.6 = – 0.35$ m

$y' = (20491 + 148) / 9841 = 2.10$ m $e_y = 0.5 \times 6 – 0.75 – 2.1 = 0.15$ m

$Mx' = 9841 \times 0.15 = 1476$ kNm $My' = 9841 \times (–) 0.35 = – 3444$ kNm

$p_1 = 9841 / (12 \times 6) – 1476 \times 6 / (12 \times 6^2) –3444 \times 6 / (6 \times 12^2) = 140$ kN/m²

$p_2 = 9841 / (12 \times 6) – 1476 \times 6 / (12 \times 6^2) + (–) 3444 \times 6 / (6 \times 12^2) = 92$ kN/m²

$p_3 = 9841 / (12 \times 6) + 1476 \times 6 / (12 \times 6^2) – (–) 3444 \times 6 / (6 \times 12^2) =181$ kN/m²

$p_4 = 9841 / (12 \times 6) + 1476 \times 6 / (12 \times 6^2) + (–) 3444 \times 6 / (6 \times 12^2) = 133$ kN/m²

TABLE 5.44
Load Case 2

D+L+O+T (Dead load + Live load + Operating load + Torque)+ SCF (Short circuit force).

Item/	Magnitude					Location		Load	Pedestal size	
Load	P	Sx	My	Sy	Mx	Xn	Yn	Elev (RL)	Length [yy]	Width [xx]
Point	kN	kN	kNm	kN	kNm	m	m	M	m	m
1. C1 TG COL S-E	648		38		17	0	0	102.2	1	1
2. C2 TG COL SOUTH	1349		83		2	5.05	0	102.2	1	1.2
3. C3 TG COL S-W	973		103		32	10.4	0	102.2	1	1.2
4. C4 TG COL N-E	647		44		15	0	4.3	102.2	1	1
5. C5 TG COL NORTH	807		74		35	5.05	4.3	102.2	1	1.2
6. C6 TG COL N-W	439		92		39	10.4	4.3	102.2	1	1.2
7. COND PED G	270	0	0	−75	0	2.343	0.425	104.3	0.7	2
8. COND PED H	270	0	0	75	0	2.343	4.425	104.3	0.7	2
9. GEN.COOLER J1	12	0	0	0	0	7	0.138	104.7	0.3	2
10. GEN.COOLER J2	12	0	0	0	0	9	0.138	104.7	0.3	2
11. GEN.COOLER J3	12	0	0	0	0	7	0.511	104.7	0.3	2
12. GEN.COOLER J4	12	0	0	0	0	9	0.511	104.7	0.3	2
13. GEN COOLER DUCT	594	0	0	0	0	13	1.765	102.2	3.3	4.15
14. MISC	0	0	0	0	0	0	0	0	0	0
15. BACKFILL.	1296	0	0	0	0	5	2.15	102.2	0	0
16. RAFT	2160	0	0	0	0	5	2.15	101	0	0

TABLE 5.45
CG of Load Case 2

D+L+O+T (Dead load + Live load + Operating load + Torque)+ SCF (Short circuit force).

Item/ Load point	Ped wt P1 kN	Sum P P+P1 kN	SumP*Xn kNm	My+Sx*H kNm	x′ m	SumP*Yn kNm	Mx+Sy*H kNm	y′ m	Resultant forces R kN	Mx′ kNm	My′ kNm
1. C1 TG COL S-E	0.00	648	0	38	5.60	0	17	1.89	9802	3529	–3431
2. C2 TG COL SOUTH	0.00	1349	6812	83		0	2				
3. C3 TG COL S-W	0.00	973	10119	103		0	32		Soil bearing pressure		
4. C4 TG COL N-E	0.00	647	0	44		2782	15		p1	111	kN/m².
5. C5 TG COL NORTH	0.00	807	4075	74		3470	35		p2	63	kN/m².
6. C6 TG COL N-W	0.00	439	4566	92		1888	39		p3	209	kN/m².
7. COND PED G	75.25	345	809	0		147	–251		p4	161	kN/m².
8. COND PED H	75.25	345	809	0	e_x	1528	251.3	e_y	pavg	136	kN/m².
9. GEN.COOLER J1	37.50	50	328	0	**(–) 0.35**	7	0	**0.36**		**<SBP; Safe.**	
10. GEN.COOLER J2	37.50	50	437	0	m	7	0	m			
11. GEN.COOLER J3	37.50	50	328	0		25	0				
12. GEN.COOLER J4	37.50	50	437	0		25	0				
13. GEN COOLER DUCT	0.00	594	7769	0		1049	0				
14. MISC	0.00	0	0	0		0	0				
15. BACKFILL	0.00	1296	6739	0		2786	0				
16. RAFT	0.00	2160	11232	0		4644	0				
Sum		9802	54460	434		18358	140				

Note: $\mathbf{x'} = (54460 + 434) / 9802 = 5.6$ m $\mathbf{e_x} = 0.5 \times 12 – 0.75 – 5.6 = – 0.35$ m

$\mathbf{y'} = (18358 + 140) / 9802 = 1.89$ m $\mathbf{e_y} = 0.5 \times 6 – 0.75 – 1.88 = 0.36$ m

$R = \Sigma P = 9802$ kN

$Mx' = 9802 \times 0.36 = 3529$ kNm $My' = 9802 \times (–) 0.35 = –3431$ kNm

TABLE 5.46
Load Case 3

D+L+O+T (Dead load + Live load + Operating load + Torque) − SCF (Short circuit force).

Item / Load Point	P	Sx	My	Sy	Mx	Xn	Yn	Load Elev (RL)	Pedestal size Length [yy]	Pedestal size Width [xx]
	Magnitude					**Location**		**Load**	**Pedestal size**	
	kN	kN	kNm	kN	kNm	m	m	m	m	m
1. C1 TG COL S-E	626		35		37	0	0	102.2	1	1
2. C2 TG COL SOUTH	860		14		11	5.05	0	102.2	1	1.2
3. C3 TG COL S-W	570		19		38	10.4	0	102.2	1	1.2
4. C4 TG COL N-E	668		33		8	0	4.3	102.2	1	1
5. C5 TG COL NORTH	1296		22		24	5.05	4.3	102.2	1	1.2
6. C6 TG COL N-W	882		11		33	10.4	4.3	102.2	1	1.2
7. COND PED G	270	0	0	−75	0	2.343	0.425	104.3	0.7	2
8. COND PED H	270	0	0	75	0	2.343	4.425	104.3	0.7	2
9. GEN.COOLER J1	12	0	0	0	0	7	0.138	104.7	0.3	2
10. GEN.COOLER J2	12	0	0	0	0	9	0.138	104.7	0.3	2
11. GEN.COOLER J3	12	0	0	0	0	7	0.511	104.7	0.3	2
12. GEN.COOLER J4	12	0	0	0	0	9	0.511	104.7	0.3	2
13. GEN COOLER DUCT	594	0	0	0	0	13	1.765	102.2	3.3	4.15
14. MISC	0	0	0	0	0	0	0	0	0	0
15. BACKFILL	1296	0	0	0	0	5	2.15	102.2	0	0
16. RAFT	2160	0	0	0	0	5	2.15	101	0	

TABLE 5.47
CG of Load Case 3

D+L+O+T (Dead load + Live load + Operating load + Torque) – SCF (Short circuit force).

Item /	Ped wt	Sum P							Resultant forces		
Load	P1	P+P1	SumP*Xn	My+Sx*H	x′	SumP*Yn	Mx+Sy*H	y′	R	Mx′	My′
point	kN	kN	kNm	kNm	m	kNm	kNm	M	kN	kNm	kNm
1. C1 TG COL S-E	0.00	626	0	35	5.59	0	37	2.30	9840	–492	–3346
2. C2 TG COL SOUTH	0.00	860	4343	14		0	11				
3. C3 TG COL S-W	0.00	570	5928	19		0	38		Soil bearing pressure		
4. C4 TG COL N-E	0.00	668	0	33		2872	8		p1	167	kN/m².
5. C5 TG COL NORTH	0.00	1296	6545	22		5573	24		p2	120	kN/m².
6. C6 TG COL N-W	0.00	882	9173	11		3793	33		p3	153	kN/m².
7. COND PED G	75.25	345	809	0		147	–251		p4	107	kN/m².
8. COND PED H	75.25	345	809	0	e_x	1528	251.3	e_y	pavg	137	kN/m².
9. GEN.COOLER J1	37.50	50	328	0	(–) **0.34**	7	0	**–0.05**	< SBP; Safe		
10. GEN.COOLER J2	37.50	50	437	0	m	7	0	m			
11. GEN.COOLER J3	37.50	50	328	0		25	0				
12. GEN.COOLER J4	37.50	50	437	0		25	0				
13. GEN COOLER DUCT	0.00	594	7769	0		1049	0				
14. MISC	0.00	0	0	0		0	0				
15. BACKFILL	0.00	1296	6739	0		2786	0				
16. RAFT	0.00	2160	11232	0		4644	0				
Sum		9840	54876	134		22456	151				

Note: **x′** = (54876 + 134) / 9840 = 5.59 m e_x = 0.5 × 12–0.75 – 5.59 = – 0.34 m
y′ = (22456 + 151) / 9840 = 2.30 m e_y = 0.5 × 6–0.75–2.297 = –0.05 m
R = Σ P = 9840 kN
Mx′ = 9802 × (–) 0.05 = –492 kNm My′ = 9840 × (–) 0.34 = –3346 kNm

TABLE 5.48
Load Case 4

D+L+O+T (Dead load + Live load + Operating load + Torque) + FAIL LD (Failure load).

Item /	Magnitude					Item /		Magnitude		
Load	P	Sx	My	Sy	Mx	Load		P	Sx	My
Point	kN	kN	kNm	kN	kNm	point		kN	kN	kNm
1. C1 TG COL S-E	655		29		25	0		−236	−394	−551
2. C2 TG COL SOUTH	1169		32		5	5.05	0	102.2	1	1.2
3. C3 TG COL S-W	842		27		35	10.4	0	102.2	1	1.2
4. C4 TG COL N-E	680		33		2	0	4.3	102.2	1	1
5. C5 TG COL NORTH	1096		33		33	5.05	4.3	102.2	1	1.2
6. C6 TG COL N-W	718		25		35	10.4	4.3	102.2	1	1.2
7. COND PED G	270	0	0	−75	0	2.343	0.425	104.3	0.7	2
8. COND PED H	270	0	0	75	0	2.343	4.425	104.3	0.7	2
9. GEN.COOLER J1	12	0	0	0	0	7	0.138	104.7	0.3	2
10. GEN.COOLER J2	12	0	0	0	0	9	0.138	104.7	0.3	2
11. GEN.COOLER J3	12	0	0	0	0	7	0.511	104.7	0.3	2
12. GEN.COOLER J4	12	0	0	0	0	9	0.511	104.7	0.3	2
13. GEN COOLER DUCT	594	0	0	0	0	13	1.765	102.2	3.3	4.15
14. MISC	0	0	0	0	0	0	0	0	0	0
15. BACKFILL	1296	0	0	0	0	5	2.15	102.2	0	0
16. RAFT	2160	0	0	0	0	5	2.15	101	0	0

TABLE 5.49
CG of Load Case 4

D+L+O+T (Dead load + Live load + Operating load + Torque) + FAIL LD. (Failure load).

Item / Load point	Ped wt P1 kN	Sum P P+P1 kN	SumP*Xn kNm	My+Sx*H kNm	x′ m	SumP*Yn kNm	Mx+Sy*H kNm	y′ m	Resultant forces R kN	Mx′ kNm	My′ kNm
1. C1 TG COL S-E	0.00	655	0	29	5.62	0	25	2.09	10099	1616	–3737
2. C2 TG COL SOUTH	0.00	1169	5903	32		0	5				
3. C3 TG COL S-W	0.00	842	8757	27		0	35		Soil bearing pressure		
4. C4 TG COL N-E	0.00	680	0	33		2924	2		p1	144	kN/m^2
5. C5 TG COL NORTH	0.00	1096	5535	33		4713	33		p2	92	kN/m^2
6. C6 TG COL N-W	0.00	718	7467	25		3087	35		p3	189	kN/m^2
7. COND PED G	75.25	345	809	0		147	–251		p4	137	kN/m^2
8. COND PED H	75.25	345	809	0	e_x	1528	251	e_y	pavg	140	kN/m^2
9. GEN.COOLER J1	37.50	50	328	0	**(–) 0.37**	7	0	**0.16**		**<SBP;safe**	
10. GEN.COOLER J2	37.50	50	437	0	m	7	0	m			
11. GEN.COOLER J3	37.50	50	328	0		25	0				
12. GEN.COOLER J4	37.50	50	437	0		25	0				
13. GEN COOLER DUCT	0.00	594	7769	0		1049	0				
14. MISC	0.00	0	0	0		0	0				
15. BACKFILL	0.00	1296	6739	0		2786	0				
16. RAFT	0.00	2160	11232	0		4644	0				
Sum		10099	56550	179		20942	135				

Note: **x′ =** (56550 + 179) / 10099 = 5.62 m **e_x** = 0.5 × 12–0.75 – 5.62 = – 0.37 m
y′ = (20942 + 135) / 10099 = 2.09 m **e_y** = 0.5 × 6–0.75–2.09 = 0.16 m
R = Σ P = 10099 kN
Mx′ = 10099 × 0.16= 1616 kNm My′ = 10099 × (–) 0.37 = –3737 kNm

TABLE 5.50
Load Case 5

D+L+O+T (Dead load + Live load + Operating load + Torque) + Seismic load.

Item /	Magnitude					Location		Load	Pedestal size	
load	P	Sx	My	Sy	Mx	Xn	Yn	Elev (RL)	Length [yy]	Width [xx]
point	kn	kn	knm	kn	knm	m	m	m	m	m
1. C1 TG COL S-E	714		165		127	0	0	102.15	1	1
2. C2 TG COL SOUTH	722		197		204	5.05	0	102.15	1	1.2
3. C3 TG COL S-W	426		167		188	10.4	0	102.15	1	1.2
4. C4 TG COL N-E	396		165		115	0	4.3	102.15	1	1
5. C5 TG COL NORTH	672		197		212	5.05	4.3	102.15	1	1.2
6. C6 TG COL N-W	376		167		185	10.4	4.3	102.15	1	1.2
7. COND PED G	270	0	0	–75	0	2.343	0.425	104.3	0.7	2
8. COND PED H	270	0	0	75	0	2.343	4.425	104.3	0.7	2
9. GEN. COOLER J1	12	0	0	0	0	7	0.138	104.65	0.3	2
10. GEN. COOLER J2	12	0	0	0	0	9	0.138	104.65	0.3	2
11. GEN. COOLER J3	12	0	0	0	0	7	0.511	104.65	0.3	2
12. GEN. COOLER J4	12	0	0	0	0	9	0.511	104.65	0.3	2
13. GEN COOLER DUCT	594	0	0	0	0	13	1.765	102.15	3.3	4.15
14. MISC	0	0	0	0	0	0	0	0	0	0
15. BACKFILL	1296	0	0	0	0	5	2.15	102.15	0	0
16. RAFT	2160	0	0	0	0	5	2.15	100.95	0	0

TABLE 5.51
CG Load Case 5

D+L+O+T (Dead load + Live load + Operating load + Torque) + Seismic load.

Item / load point	Ped wt P1 kN	Sum P P+P1 kN	SumP*Xn kNm	My+Sx*H kNm	x′ m	SumP*Yn kNm	Mx+Sy*H kNm	y′ m	Resultant forces R kN	Mx′ kNm	My′ kNm
1. C1 TG COL S-E	0.00	714	0	165	5.50	0	127	2.12	8245	1072	–2061
2. C2 TG COL SOUTH	0.00	722	3646	197		0	204				
3. C3 TG COL S-W	0.00	426	4430	167		0	188		Soil bearing pressure		
4. C4 TG COL N-E	0.00	396	0	165		1703	115		p1	114	kN/m²
5. C5 TG COL NORTH	0.00	672	3394	197		2890	212		p2	85	kN/m²
6. C6 TG COL N-W	0.00	376	3910	167		1617	185		p3	144	kN/m²
7. COND PED G	75.25	345	809	0		147	–251.3		p4	115	kN/m²
8. COND PED H	75.25	345	809	0	e_x	1528	251.25	e_y	pavg	115	kN/m²
9. GEN. COOLER J1	37.50	50	328	0	(–) **0.25**	7	0	**0.13**		**< SBP; safe**	
10. GEN. COOLER J2	37.50	50	437	0	m	7	0	m			

11. GEN. COOLER J3	37.50	50	328	0	25	0
12. GEN. COOLER J4	37.50	50	437	0	25	0
13. GEN COOLER DUCT	0.00	594	7769	0	1049	0
14. MISC	0.00	0	0	0	0	0
15. BACKFILL	0.00	1296	6739	0	2786	0
16. RAFT	0.00	2160	11232	0	4644	0
Sum		8245	44268	1058	16427	1031

Note: $x' = (44268 + 1058) / 8245 = 5.5$ m $e_x = 0.5 \times 12 - 0.75 - 5.5 = -0.25$ m

$y' = (16427 + 1031) / 8245 = 2.12$ m $e_y = 0.5 \times 6 - 0.75 - 2.12 = 0.13$ m

$R = \Sigma P = 8245$ kN

$Mx' = 8245 \times 0.13 = 1072$ kNm $My' = 8245 \times (-) 0.25 = -2061$ kNm

TABLE 5.52
Load Case 6

DL +LL+ OPTG LOAD

Item / load point	Magnitude					Location			Pedestal size	
	P	Sx	My	Sy	Mx	Xn	Yn	Load Elev(RL)	Length [yy]	Width [xx]
	kN	kN	kNm	kN	kNm	m	M	m	m	m
1. C1 TG COL S-E	635	5	18	8	12	0	0	102.15	1	1
2. C2 TG COL SOUTH	1104	2	23	10	4	5.05	0	102.15	1	1.2
3. C3 TG COL S-W	771	7	23	10	14	10.4	0	102.15	1	1.2
4. C4 TG COL N-E	657	2	18	8	4	0	4.3	102.15	1	1
5. C5 TG COL NORTH	987	7	23	10	16	5.05	4.3	102.15	1	1.2
6. C6 TG COL N-W	640	7	23	10	14	10.4	4.3	102.15	1	1.2
7. COND PED G	270	0	0	−75	0	2.343	0.425	104.3	0.7	2
8. COND PED H	270	0	0	75	0	2.343	4.425	104.3	0.7	2
9. GEN. COOLER J1	12	0	0	0	0	7	0.138	104.65	0.3	2
10. GEN. COOLER J2	12	0	0	0	0	9	0.138	104.65	0.3	2
11. GEN. COOLER J3	12	0	0	0	0	7	0.511	104.65	0.3	2
12. GEN. COOLER J4	12	0	0	0	0	9	0.511	104.65	0.3	2
13. GEN COOLER DUCT	594	0	0	0	0	13	1.765	102.15	3.3	4.15
14. MISC	0	0	0	0	0	0	0	0	0	0
15. BACKFILL	1296	0	0	0	0	5	2.15	102.15	0	0
16. RAFT	2160	0	0	0	0	5	2.15	100.95	0	0

TABLE 5.53
CG of Load Case 6

DL +LL+ OPTG LOAD

Item / Load point	Ped wt P1 kN	Sum P P+P1 kN	SumP*Xn kNm	My+Sx*H kNm	x′ m	SumP*Yn kNm	Mx+Sy*H kNm	y′ m	Resultant forces R kN	Mx′ kNm	My′ kNm
1. C1 TG COL S-E	0.00	635	0	24	5.58	0	21.4	2.07	9733	1752	−3212
2. C2 TG COL SOUTH	0.00	1104	5575	25.4		0	16				
3. C3 TG COL S-W	0.00	771	8018	31.4		0	26		Soil bearing pressure		
4. C4 TG COL N-E	0.00	657	0	20.4		2825	13.6		p1	133	kN/m²
5. C5 TG COL NORTH	0.00	987	4984	31.4		4244	28		p2	89	kN/m²
6. C6 TG COL N-W	0.00	640	6656	31.4		2752	26		p3	182	kN/m²
7. COND PED G	75.25	345	809	0		147	−251.3		p4	137	kN/m²
8. COND PED H	75.25	345	809	0	e_x	1528	251.25	e_y	pavg	135	
9. GEN.COOLER J1	37.50	50	328	0	**(−) 0.33**	7	0	**0.18**		**< SBP; Safe.**	
10. GEN.COOLER J2	37.50	50	437	0	m	7	0	m			
11. GEN.COOLER J3	37.50	50	328	0		25	0				
12. GEN.COOLER J4	37.50	50	437	0		25	0				
13. GEN COOLER DUCT	0.00	594	7769	0		1049	0				
14. MISC	0.00	0	0	0		0	0				
15. BACK FILL.	0.00	1296	6739	0		2786	0				
16. RAFT	0.00	2160	11232	0		4644	0				
Sum		9733	54121	164		20039	131				

Note: **x′ =** (54121 + 164) / 9733 = 5.58 m $\mathbf{e_x}$ = 0.5 × 12–0.75 – 5.58 = – 0.33

my′ **=** (20039 + 131) / 9733 = 2.07 m $\mathbf{e_y}$ = 0.5 × 6–0.75–2.07 = 0.18 m

R = Σ P = 9733 kN

Mx′ = 9733 × 0.18= 1752 kNm My′ = 9733 × (–) 0.33 = −3212 kNm

TABLE 5.54
Load Case 7

DL +LL+ OPTG LOAD + Seismic load

Item /	Magnitude					Location		Load	Pedestal size	
Load	P	Sx	My	Sy	Mx	Xn	Yn	Elev (RL)	length[yy]	wide [xx]
point	kN	kN	kNm	kN	kNm	m	m	m	m	m
1. C1 TG COL S-E	560	32	165	28	101	0	0	102.15	1	1
2. C2 TG COL SOUTH	1006	51	197	34	196	5.05	0	102.15	1	1.2
3. C3 TG COL S-W	686	33	167	26	188	10.4	0	102.15	1	1.2
4. C4 TG COL N-E	579	28	130	43	108	0	4.3	102.15	1	1
5. C5 TG COL NORTH	891	55	152	53	180	5.05	4.3	102.15	1	1.2
6. C6 TG COL N-W	555	32	123	44	184	10.4	4.3	102.15	1	1.2
7. COND PED G						2.343	0.425	104.3	0.7	2
8. COND PED H						2.343	4.425	104.3	0.7	2
9. GEN.COOLER J1						6.625	0.138	104.65	0.3	2
10. GEN.COOLER J2						8.825	0.138	104.65	0.3	2
11. GEN.COOLER J3						6.625	0.511	104.65	0.3	2
12. GEN.COOLER J4						8.825	0.511	104.65	0.3	2
13. GEN COOLER DUCT						13.075	1.765	102.15	3.3	4.15
14. MISC						0	0	0	0	0
15. BACK FILL.	0					5.2	2.15	102.15	0	0
16. RAFT	2160					5.2	2.15	100.95	0	0

TABLE 5.55
CG of Load Case 7

DL +LL+ OPTG LOAD + Seismic load

Item / Load point	Ped wt P1	Sum P P+P1	SumP*Xn	My+Sx*H	x′	SumP*Yn	Mx+Sy*H	y′	Resultant forces R	Mx′	My′
	kN	kN	kNm	kNm	m	kNm	kNm	m	kN	kNm	kNm
1. C1 TG COL S-E	0.00	560	0	203.4	5.41	0	134.6	2.23	6738	135	−1078
2. C2 TG COL SOUTH	0.00	1006	5080	258.2		0	236.8				
3. C3 TG COL S-W	0.00	686	7134	206.6		0	219.2		Soil bearing pressure		
4. C4 TG COL N-E	0.00	579	0	163.6		2490	159.6		p1	99	kN/m^2
5. C5 TG COL NORTH	0.00	891	4500	218		3831	243.6		p2	84	kN/m^2
6. C6 TG COL N-W	0.00	555	5772	161.4		2387	236.8		p3	103	kN/m^2
7. COND PED G	75.25	75	176	0		32	0		p4	88	kN/m^2
8. COND PED H	75.25	75	176	0	e_x	333	0	e_y	pavg	94	kN/m^2
9. GEN.COOLER J1	37.50	38	248	0	**(–) 0.16**	5	0	**0.02**		**< SBP; Safe.**	
10. GEN.COOLER J2	37.50	38	331	0	m	5	0	m			
11. GEN.COOLER J3	37.50	38	248	0		19	0				
12. GEN.COOLER J4	37.50	38	331	0		19	0				
13. GEN COOLER DUCT	0.00	0	0	0		0	0				
14. MISC	0.00	0	0	0		0	0				
15. BACKFILL	0.00	0	0	0		0	0				
16. RAFT	0.00	2160	11232	0		4644	0				

(Continued)

TABLE 5.55 ***(Continued)***
CG of Load Case 7

Item / Load point	Ped wt P1 (kN)	Sum P P+P1 (kN)	SumP*Xn (kNm)	My+Sx*H (kNm)	x′ (m)	SumP*Yn (kNm)	Mx+Sy*H (kNm)	y′ (m)	Resultant forces R (kN)	Mx′ (kNm)	My′ (kNm)
Sum		6738	35230	1211		13765	1231				

Note: x′ = (35230 + 1211) / 6738 = 5.41 m e_x = 0.5 × 12–0.75 – 5.41 = – 0.16 m

y′ = (13765 + 1231) / 6738 = 2.23 m e_y = 0.5 × 6–0.75–2.23 = 0.02 m

R = Σ P = 6738 kN

Mx′ = 6738 × 0.02= 135 kNm My′ = 6738 × (–) 0.16 = −1078 kNm

5.5.1.3 Summary of observations

TABLE 5.56
Soil bearing pressure:

		Load CG		Gross bearing Pressure					
		e_x	e_y	p_1	p_2	p_3	p_4	pavg	
Load Case	**Load Description**	**m**	**m**	**kN/m²**	**kN/m²**	**kN/m²**	**kN/m²**	**kN/m²**	
1	D+L+O+T	–0.35	0.15	140	92	181	137	137	Safe.
2	D+L+O+T + SCF	–0.35	0.36	111	63	209	161	136	Safe.
3	D+L+O+T – SCF	–0.34	–0.05	167	120	153	107	137	Safe.
4	D+L+O+T+ FL	–0.37	0.16	144	92	189	137	140	Safe.
5	D+L+O+T+SL	–0.25	0.13	114	85	144	115	115	Safe.
6	DL +LL+ O	–0.33	0.18	133	89	182	137	135	Safe.
7	DL +LL+ O + SL	–0.16	0.02	99	84	103	88	94	Safe.

80% of Allowable Gross bearing pressure = $0.8 \times 350 = 280$ kN/m^2 ($0.8 \times$ SBP)

Eccentricity of Foundation Load CG:

Maximum value of $e_x = 0.33$ m	$< 0.05 \times 12 = 0.6$ m	Safe (ACI 351.3 R-04)
$< 0.03 \times 12 = 0.36$ m		Safe (IS: 2974.3 1992)
Maximum value of $e_y = 0.18$ m	$< 0.05 \times 6 = 0.3$ m	Safe (ACI 351.3 R-04)
$< 0.03 \times 6 = 0.18$ m		Safe (IS: 2974.3 1992)

Minimum Thickness and Rigidity of Raft:

Minimum recommended thickness should be as follows:

i) $0.07\ L^{4/3} = 0.07 \times 4.2^{4/3} = 0.474$ m

L = average of two adjacent clear span = 4.2 m
Span 1 = $5.05 - (0.5 \times 1 + 0.5 \times 1) = 4.05$ m
Span 2 = $5.35 - (0.5 \times 1 + 0.5 \times 1) = 4.35$ m

ii) $1/5\ B = 1/5 \times 6 = 1.2$ m
iii) $1/10\ L = 1/10 \times 12 = 1.2$ m

D provided = 1.2 m > = D minimum; Hence, safe.

Rigidity Check:

As per IS: 2950 (Part I) – 1981 Appendix – C

For the rectangular raft:
A. Cl.no 5.1.1 a relative stiffness factor, $K > 0.5$
Or
B. Column spacing < 1.75 / 1

A. Cl.no 5.1.1 a relative stiffness factor, K > 0.5

$K = [E / 12\ Es] \times [d/b]^3 = 0.633 > 0.5$; hence, rigid foundation
Where, E = 285000 kg/cm^2 for concrete M25
(E = $5700 \sqrt{fck} = 5700 \times \sqrt{25} = 28500$ N/mm^2]
Es = 300 kg/cm^2 for soil
Raft thickness, d = 120 cm
Length of the section in the bending axis, b = 600 cm

B. Column spacing < 1.75 / λ

Longitudinal direction:

$1.75 / \lambda = 1427$ cm   actual spacing = 430 cm < 1427, hence Rigid.
Column spacing = 430 cm
$\lambda = {}^{0.25}\sqrt{[kB / (4Ec.I)]} = 0.001$
$k = 0.65 \times [{}^{1/12}\sqrt{(Es.B^4/EI)}] \times [Es / (1 - \mu^2)] \times [1 / B] = 0.371$ kg/cm^3
k = modulus of subgrade reaction
B = width of foundation = 600 cm
$I = B\ d^3 / 12 = 86400000$ cm^4 $\mu = 0.3$

Transverse direction

$1.75 / \lambda = 1626$ cm   actual spacing = 535 cm < 1626; hence, rigid.
Column spacing = 535 cm
$\lambda = {}^{0.25}\sqrt{[kB / (4Ec.I)]} = 0.001$
$k = 0.65 \times [{}^{1/12}\sqrt{(Es.B^4/EI)}] \times [Es / (1 - \mu^2)] \times [1 / B] = 0.371$ kg/cm^3
B = 1200 cm
$I = B\ d^3 / 12 = 172800000$ cm^4 $\mu = 0.3$

Strength design:

i) Soil bearing pressure for Dead load + Live load including operating load = 135 kN/m^2
ii) Maximum soil bearing pressure for other combinations including incidental loading
= 140 kN/m^2

Considering allowable increase in SBP for incidental load combination, the equivalent SBP to compare with DL + LL combination should be equal to 140 / 1.25 = 112 kN/m^2

So, design SBP = 135 − self-weight of raft = 135−1.2 × 25 = 105 kN/m^2 (net upward pressure for strength design)

Along the X axis

Effective span = 5.35 m   Moment = $105 \times 5.35^2 / 10 = 301$ kNm / meter width
Projection outside the column face = (12 − 10.4 − 0.5 × 1−0.5 × 1) / 2 = 0.3 m
Cantilever moment = $105 \times 0.3^2 / 2 = 5$ kNm / meter width

Along the Y axis

Effective span = 4.3 m Moment = $105 \times 4.3^2 / 10 = 194$ kNm / meter width
Projection outside the column face = (6−4.3 − 0.5 × 1−0.5 × 1) / 2 = 0.35 m
Cantilever moment = $105 \times 0.35^2 / 2 = 6$ kNm / meter width

According to IS: 456–2000, working stress method

Moment = 301 kNm/m M25   grade of concrete
$\sigma bc = 8.5$ N/mm^2
$\sigma st = 230$ MPa
b = 1000 mm

$j = 0.87$
$k = 0.3$

Effective depth = d
d required = $\sqrt{(M/q.b)} = \sqrt{[301000000 / (1.11 \times 1000)]}$ = 521 mm
$q = 1/2\ \sigma cb.\ k.j = 0.5 \times 8.5 \times 0.3 \times 0.87 = 1.11$
d provided = 1200–75 = 1125 mm > d required
Ast required = M / (σst. j. d) = $301000000 / (230 \times 0.87 \times 1125)$
= 1337 mm^2 /m
Ast min = 0.0012 Ag = $0.0012 \times 1000 \times 1200$ = 1440 mm^2 /m
Provide reinforcement bar 25 mm diameter @ 300 mm C/C (= 1637 mm^2 /m)
Ast provided = 1637 mm^2 /m > Ast reqd. Safe.

Along the Y axis

Moment = 194 kNm/m
Ast required = M / (σst. j. d) = $194000000 / (230 \times 0.87 \times 1125)$
= 862 mm^2 /m
Provide reinforcement bar 25 mm diameter @ 300 mm C/C (= 1637 mm^2 /m)
Ast provided = 1637 mm^2 /m > Ast reqd. safe.

Shear strength:

The critical section for shear will be around corner columns and the columns at centers

So let us check the shear strength of the raft for both the cases as below.

Corner column

Maximum column load = 973 kN
[Load case 2; Col C3]

There are two free edges of the raft. The critical section of shear in the raft is shown on the opposite face of free edges. The critical section for shear shall be at a distance d/2 from the periphery of the column, perpendicular to the plane of the raft where d is the effective depth.

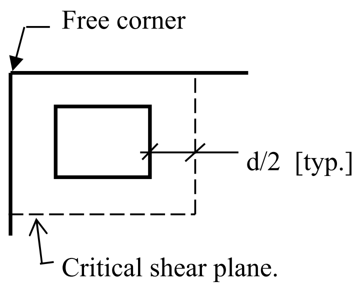

FIGURE 5.42 Shear at corner column

Raft thickness = 1.2 m
Effective thickness = 1.11 m
Shear area = [(Pedestal width + edge dist + deff/2) + (Ped length + edge dist + deff/2)]. D
$= [(1 + 0.3 + 0.56) + (1 + 0.35 + 0.56)] \times 1.2$
$= 4.515$ m^2

Shear stress developed = 973000 / 4515000 = 0.22 N/mm^2
Allowable shear stress, τ_c = 0.16 $\sqrt{fck}$ = 0.80 N/mm^2 > 0.22 N/mm^2; Safe.

Center column

Maximum column load = 1349 kN
[Load case 2; Column C2]
Raft thickness = 1.2 m
Effective thickness = 1.11 m

Shear area = [(Pedestal width + deff) + (Ped length + edge dist + deff/2)]. D
= [(1 + 1.11) + (1.2 + 0.35 + 0.56)] × 1.2
= 5.06 m^2

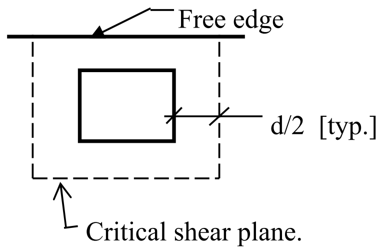

FIGURE 5.43 Shear at corner column

Shear stress developed = 1349000 / 5062500 = 0.27 N/mm^2
Allowable shear stress, τ_c = 0.16 $\sqrt{fck}$ = 0.80 N/mm^2 > 0.27 N/mm^2; Safe.

According to ACI 318–19 ***[SI Unit]***

Flexural strength:

Along the X axis
M = 301 kNm/m Load factor = 1.4 (Table 5.3.1 ACI 318 – Table 5.3.1)
Mu = 1.4 × 301 = 421 kNm ϕ = 0.9
Refer to Figure 5.55 for stress and strain diagrams for the slab.
f'c = 20 N/mm^2 (cylinder strength) fy = 415 MPa
bw = 1000 mm d = 1113 mm As = 1637 mm^2
Re-bars:

Top layer	25 mm diameter bars @ 300 c/c	Ast provided = 1637 mm^2
Bottom layer	25 mm diameter bars @ 300 c/c	Ast provided = 1637 mm^2
		Total = 3274 mm^2 > Minimum re-bars,

Minimum shrinkage reinforcement = 0.0018 Ag = 0.0018 × 1200 × 1000 = 2160 mm^2

From Figure 5.55,
C = T *0.85fc'. a. bw = As.fy* *a = As.fy /(0.85 fc' bw)*
a = 1637 × 415 / (0.85 × 20 × 1000) = 40 mm
ϕ *Mn =*ϕ *As fy (d – a/2) =0.9 × 1637 × 415 × (1113 – 40 / 2) / 10*^{6} *=668kNm > Mu; safe.*

Along the Y axis
M = 194 kNm/m Load factor = 1.4 (Table 5.3.1 ACI 318 – Table 5.3.1)
Mu = 1.4 × 194 = 272 kNm ϕ = 0.9
Refer to Figure 5.55 above for stress and strain diagrams for the slab.

f'c = 20 N/mm^2 (cylinder strength) fy = 415 MPa
bw = 1000 mm d= 1113 mm As = 1637 mm^2
Re-bars:

Top layer	25 mm diameter bars @ 300 c/c	Ast provided = 1637 mm^2
Bottom layer	25 mm diameter bars @ 300 c/c	Ast provided = 1637 mm^2
		Total = 3274 mm^2 > minimum re-bars,

Minimum shrinkage reinforcement = 0.0018 Ag = 0.0018 × 1200 × 1000 = 2160 mm^2

From Figure 5.55,
C = T *0.85fc'. a. bw = As.fy* *a = As.fy /(0.85 fc' bw)*
a = 1637 × 415 / (0.85 × 20 × 1000) = 40 mm
ϕ Mn =ϕ As fy (d – a/2) =0.9 × 1637 × 415 × (1113 – 40 / 2) / 10^6 =668kNm > Mu; safe.

Shear capacity

Shear stress around the corner column = 0.22 N/mm^2
Shear stress around the center column = 0.27 N/mm^2
bw = 1000 mm d = 1200 mm
Strength reduction factor, ϕ = 0.75 ACI 318–19 (Table 21.2.1)
Vu = 0.27 × 1000 × 1200 / 1000 = 324 kN
Required nominal strength, V_n = Vu/ϕ = 432 kN ϕ Vn > = Vu ACI 318–19 (9.5.1.1)
Shear capacity = Vc
Vc = 0.17 λ √fc'. bw. d = (0.17 × 0.59 × √20 × 1000 × 1200) / 1000 = 538 kN
λ = √ [2/(1+0.004d)] = 0.59 < 1 ACI 318–19 (Table 22.5.5.1a)
ACI 318–19 (22.5.5.1.3)
Vc > Vn; safe.

5.5.2 Circular raft on the pile

5.5.2.1 Introduction

The workout example is design of an RCC circular raft supporting a steel storage tank (7m diameter× 10 m height). The raft is resting on a group of piles. The tank base is seated on a sand bitumen carpet laid over the RCC raft. The top of the raft / pile cap is above the finish-grade level.

5.5.2.2 Sketch

Design parameters

Natural-grade level (RL) NGL = 100 M
Finish-grade level (RL) FGL = 100.3 M

Groundwater level (RL) GWL = 99 M
Top of the raft/pile cap (RL) TOP = 100.6 M

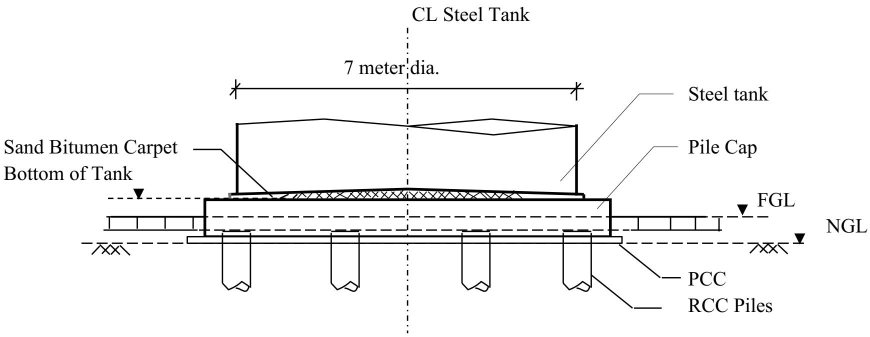

FIGURE 5.44 Elevation of tank foundation

Bottom of the tank (RL) BOT = 100.6 M
Bottom of the pile cap (RL) BOF = 100 M
(RL means plant reference level)

Pile capacity:
Pile diameter, Pdia = 0.5 m
Axial capacity, Paxial = 600 kN Shear, Pshear = 50 kN Uplift, Puplift = 120 kN

Foundation material:

Concrete grade	M25	(fck = 25 N/mm^2)
Unit weight of concrete	γconc = 25 kN/m^3	
Unit weight of soil	γsoil = 18 kN/m^3	
Reinforcement steel	fy = 415 MPa (yield stress of reinforcement bars)	

Clear cover – 100 mm at the bottom of the pile cap and 75 mm at the top and sides.
Maximum diameter of re-bars = 25 mm
Edge distance = 0.5 × 0.5 + 0.15 = 0.4 m

Design wind pressure	p_{wind} = 2.5 kN/m^2
Ground surcharge	γ_{surch} = 10 kN/m^2
Seismic coefficient	αh = 0.12
Liquid density	γ_{LQD} = 10 kN/m^3 (water storage tank)

Load on the pile group

Steel tank + fill weight
Diameter of steel tank, T_{DIA} = 7m
Height of tank, T_{HT} = 10 m
Maximum liquid height, LQD_{HT} = 9.7 m
Self-weight of the tank, Tselfwt = 258 kN

Tfullwt = Liquid weight + Tselfwt.
= [(3.14 × 7^2) / 4] × 9.7 × 10 + 258 = 3989 kN

Foundation weight

Pile cap thickness, B_{TH} = 0.6 m   Diameter of Pilecap, D_{BASE} = 8.2 m
Weight of the pile cap, W_{CAP} = 3.14 × 8.2^2 × 0.25 × 0.6 × 25 = 792 kN
Weight of the sand bitumen carpet = 3.14 × 7^2 × 0.25 × 0.0625 × 20 = 48 kN
Sand bitumen carpet (coarse sand mixed with 80/100 bitumen – 8–10% by vol.)
Average thickness = 0.5 × (70 + 50) = 62.5 mm
Weight of foundation, W_{FDN} = 792 + 48 = 840 kN.

Wind load

Wind pressure = 2.5 kN/m^2
Pressure coefficient = 0.7
Wind force, P_{wind} = 123 kN (0.7 × 2.5 × 7 × 10)
Hw = 5.60 m   [10 × 0.5 + (100.6 – 100)]
Mwind = 689 kNm   (123 × 5.6)

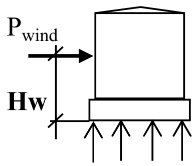

FIGURE 5.45 Shear at the corner column

Seismic load

a) *Tank empty*

Weight of the tank (empty) = 258 kN
Weight of the foundation = 840 kN
Seismic coefficient, αh = 0.12
Seismic force = 132 kN [(258 + 840) × 0.12]
Hw (tank) = 5.6 m
Mseis = 222 kNm   [258 × (5.6 + 0.6) × 0.12 + 840 × 0.5 × 0.6 × 0.12]

b) *Tank full*

Wt. of tank (full) = 3989 kN
Wt. of Fdn. = 840 kN
Seismic coefficient, αh = 0.12
Hw (tank) = 5.6 m

To determine Hydro dynamic Pressure in tank in Seismic condition

p_w = Pressure on wall
αh = 0.12
γ_{LQD} = 10 kN/m^3
R = T_{DIA} /2 = 3.5 m
h = T_{HT} = 10 m

at   y = h = 10 m $\phi = 0$ $\cos\phi = 1$
tanh √ 3 (R/h) = 0.541

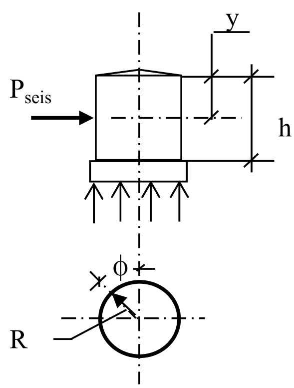

FIGURE 5.46 Seismic Load

$p_w = \alpha h.\ \gamma_{LQD}.h.\ \sqrt{3}\cos\phi\ [y/h - 0.5(y/h)^2]\ \tanh\sqrt{3}\ (R/h)]$
$= 0.12 \times 10 \times 10 \times \sqrt{3} \times 1 \times [(10 / 10) - 0.5 \times (10 / 10)^2] \times 0.541$
$= 5.622\ kN/m^2$

At $y = 0$ m $p_w = 0\ kN/m^2$
$y = 5$ m $p_w = 4.217\ kN/m^2$
$y = 10$ m $p_w = 5.622\ kN/m^2$

Average pressure = 3.28 kN/m²
Seismic force = 3.28 × 10 × 7 = 230 kN

Total seismic force acting at the base level, i.e., on the top of the pile group level,
Pseis = 230 + 840 × 0.12 = 331 kN
Moment at the base of foundation, i.e., on top of the pile group level,
Mseis = 230 × (10 / 3 + 0.5 × 0.6) + 840 × 0.5 × 0.6 × 0.12 = 934 kNm
Base area, $A_{BASE} = 52.83\ m^2$
UDL on circular base / pile group (tank full + foundation.),
$q_{TANK} = (3989 + 840) / 52.83 = 91.41\ kN/m^2$

TABLE 5.57
Foundation load

Load case	P(kN)	H (kN)	M (kNm)	Load Description
1. DL + LL (full)	4829			Wt. of tank (full) + Wt. of fdn
2. DL + WL	1098	123	689	Wt. of tank (empty) + Wt. of fdn + Max (wind)
3. DL + LL (full) + SL	4829	331	934	Wt. of tank (full) + Wt. of fdn. + Max (wind/seis)

Maximum horizontal shear = 331 kN
Lateral capacity of a single pile = 62.5 kN (Pshear × 1.25)
Minimum required nos. of the pile = 331 / 62.5 = 5
Pile provides in rows 1 & 2 = 8 + 8 = 16 Nos. Hence, okay.

The piles are arranged in two circular rows symmetrical about the center of the raft. There are eight piles in each rows equally spaced.

Pile group array in circular rows:

Spacing of piles:
Row 1
PCD = 4 m
No of piles, $n_1 = 8$
Spacing, $S_1 = 3.14 \times 4 / 8 = 1.57$ m

Row 2
PCD = 7 m

No of piles, $n_2 = 8$
Spacing, $S_2 = 3.14 \times 7 / 8 = 2.75$ m
(PCD = pitch circle diameter)

Max. allowable	Min. allowable
spcg. = 7 × P_{dia} = 3.5 m	spcg. = 2.5 × P_{dia} = 1.25 m

Cross-sectional area of a pile, $A_{pile} = 3.14 \times 0.5^2 = 0.196$ sqm.

Let us consider the piles in a row transformed into an annular ring of equal area to find out the sectional properties of the group.

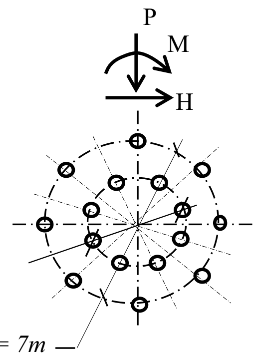

FIGURE 5.47 Pile layout

Row 1

b_1 = area of a single pile / spacing in Row 1 (S_1)
= 0.196 / 1.57 = 0.125 m
D_0 = PCD + 0.5b
= 4 + 0.5 × 0.125
= 4.063 m
D_i = PCD − 0.5b
= 4−0.5 × 0.125
= 3.938 m

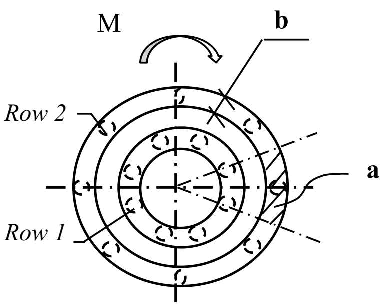

FIGURE 5.48 Annular rings of the equivalent area

$I_{row1} = 3.14\ (Do^4 - Di^4) / 64$
$= 3.14 \times (4.063^4 - 3.938^4) / 64$
$= 1.571\ m^4$

$a_1 = b \times \pi \times pcd / n_1$
$= 0.125 \times 3.14 \times 4 / 8$
$= 0.196$ m

Row 2

b_2 = Area of a single pile / spacing in Row 2 (S_2)
= 0.196 / 2.75 = 0.071 m

D_0 = PCD + 0.5b
= 7 + 0.5 × 0.071
= 7.063 m

D_i = PCD − 0.5b
= 7−0.5 × 0.071
= 6.965 m

$I_{row2} = 3.14\ (Do^4 - Di^4) / 64$
$= 3.14 \times (7.036^4 - 6.965^4) / 64$
$= 4.780\ m^4$

$a_2 = b \times \pi \times pcd / n_2$
$= 0.071 \times 3.14 \times 7 / 8$
$= 0.195$ m

$I = I_{row1} + I_{row2} = 1.571 + 4.78 = 6.351\ m^4$
$Z = I / (0.5 \times PCD) = 6.351 / 3.5 = 1.815\ m^3$

Axial force/pile:

Maximum reaction = P / n + [(M/Z) × a]
Minimum reaction = P / n − [(M/Z) × a]

Total nos. of piles = 16

Now, let us find out forces in the pile for each load cases

Load Case 1: DL + LL Wt. of tank (full) + Wt. of fdn
Axial compression = 4829 / 16 = 302 kN < 600 kN; safe.

Load Case 2: DL + WL
Axial compression = (1098 / 16) + (0.195 × 689 / 1.815)
= 68.63 + 74.02 = 143 kN < 600 kN; Safe.
Uplift = 74.02–68.63 = 5 kN (tension) < 120 kN; Safe

Load Case 3: DL + LL (full) + SL
Axial compression = (4829 / 16) + (0.195 × 934 / 1.815)
= 302 + 100 = 402 kN < 600 kN; Safe.

Uplift = 302–100 = 201 kN No Uplift.
(*) Allowable pile capacity may be increased by 25% for WL/SL combination case.

Check for lateral force:
Maximum lateral force, H = 331 kN
Total No, of piles = 8 + 8 = 16
Shear per pile = 331 / 16 = 21 kN < 50 kN; Safe

Strength design of the pile cap

Limit state method will be considered for strength design

as per the IS code IS 456: 2000

Load combination	*Pile reaction – Axial*	*Load factor*	*Factored load*
1. DL + LL (full)	302 kN	1.5	453 kN
2. DL + WL	143 kN	1.5	214 kN
3. DL + LL (full) + SL	402 kN	1.2	483 kN

(Load factors conforms to Table 18: IS 456: 2000)

Shear stress in the pile cap:

Maximum pile reaction = 483 kN (factored)

Raft thickness = 0.6 m
Edge distance = 0.4 m
Effective depth, d = 0.475 m
[(0.6 × 1000–25–100) / 1000]

Shear area = [(0.5 + 0.475) + 2 × (0.4 + 0.5 × 0.5 + 0.5 × 0.475)] × 0.6 = 1.65 sqm.

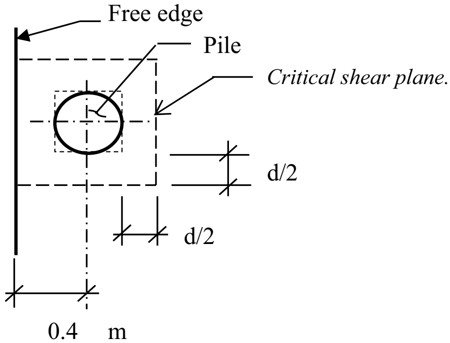

FIGURE 5.49 Critical section for shear

Shear stress developed = 483 / 1.65 = 293 kN/m^2 = 0.293 N/mm^2 < Allowable shear; safe.

Allowable shear stress, τc:

$\tau c = 0.25\sqrt{fck} = 0.25 \times \sqrt{25} = 1.25$ N/mm^2 [IS 456:2000 31.6.2]

5.5.2.3 Flexural strength

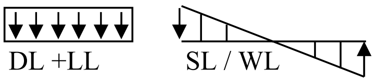

FIGURE 5.50 Uniform base pressure

Pressure on the pile group

DL + LL (full): P/A p_1 = 4829 / 52.83 = 91.41 kN/m^2
SL: (+ / −) M / Z p_2 = 934 / 54.15 = 17.25 kN/m^2
WL: (+ / −) M / Z p_3 = 689 / 54.15 = 12.72 kN/m^2
Self-weight of the raft: p_4 = 0.6 × 25 = 15.00 kN/m^2

where A = 52.83 sqm (= A_{BASE}) and Z = 3.14 × 8.2^3 / 32 = 54.15 m^3
Dbase = 8.2 m

Net upward pressure, p_{design}:

DL + LL + SL/WL = 91.41 + 17.25−15 = 93.66 kN/m^2; LF = 1.2
Factored pressure = 93.66 × 1.2 = 112 kN/m^2

DL + LL = 91.41−15 = 76.41 kN/m^2 LF = 1.5
Factored pressure = 76.41 × 1.5 = 115 kN/m^2

So, p_{design} = 115 kN/m^2

Case 1: When re-bars are placed in radial and circumferential directions

Moment radial (factored):
Span = 4.0 m [PCD of Row1 = 4 m; PCD of Row2 − Row 1= 3m]
Mu_rad = 115 × 4^2 /10 = 184 kNm
Mu_rad/bd^2 = 184 × 10^6 / (1000 x 475^2) = 0.82
pt = 0.246 % and pc = 0.2 % [from SP16 Table 3 & 50]

Ast required,
Ast = 0.246 × 1000 × 475 / 100 = 1169 mm^2
Asc = 0.20 × 1000 × 475 / 100 = 950 mm^2
Provide bars along the radial direction at top and bottom faces:
25 mm diameter bars @ 250 c/c; Ast provided = 1964 mm^2 > Ast required; safe.

Moment circumferential

Span = 2.75 m [spacing of piles, S1 = 1.57 m and S2 = 2.75m; π. PCD/no of piles]
$Mu\_circ = 115 \times 2.75^2 / 10 = 87$ kNm
$Mu\_circ/bd^2 = 87 \times 10^6 / (1000 \times 475^2) = 0.39$
pt = 0.113 % and pc = 0.20 % [from SP16 Table 3 & 50]
Ast required:
$Ast = 0.113 \times 1000 \times 475 / 100 = 537\ mm^2$
$Asc = 0.20 \times 1000 \times 475 / 100 = 950\ mm^2$
Provide bars along the circumferential direction at top and bottom faces:
20 mm diameter bars @ 300 c/c; Ast provided = 1048 mm^2 > Ast required; Safe.

Case 2: Alternate to Case 1, when re-bars are placed in the orthogonal direction

Mortho = [Mu_rad + Mu_circ] / 1.414 = (184 + 87) / 1.414 = 192 kNm
$Mu\_ortho/bd^2 = 192 \times 10^6 / (1000 \times 475^2) = 0.85$
pt = 0.261 % and pc = 0.20 % [from SP16 Table 3 & 50]
Ast required,
$Ast = 0.261 \times 1000 \times 475 / 100 = 1240\ mm^2$
$Asc = 0.20 \times 1000 \times 475 / 100 = 950\ mm^2$

Provide bars along the orthogonal direction at top and bottom faces:
25 mm diameter bars @ 275 c/c; Ast provided = 1786 mm^2 > Ast required; safe.

Strength design of pile cap – ACI 318–19

Foundation material:

Concrete grade
$fc' = 20\ N/mm^2$
fy = 450 MPa
Dead load, D = Tank Self weight + Foundation = 258 + 840 = 1098 kN
Liquid fill, $L = [(3.14 \times 7^2) / 4) \times 9.7 \times 10] = 3731$ kN

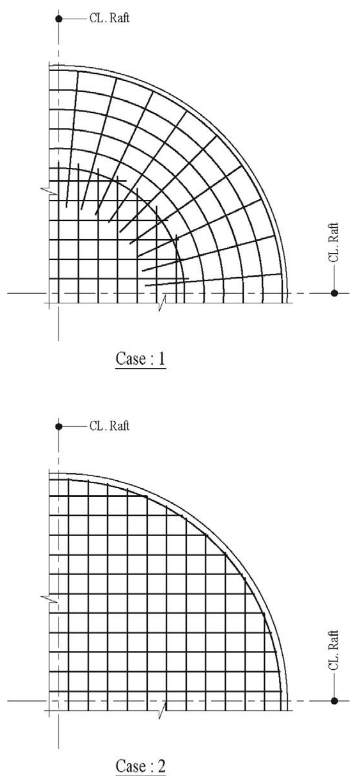

FIGURE 5.51 Detail of reinforcement in a circular raft

Wind load, W: $M_w = 689$ kNm $H_w = 123$ kN
Seismic load, E: $M_E = 934$ kNm $H_E = 331$ kN

TABLE 5.58
Foundation load

Load case (#)	P(kN)	H (kN)	M (kNm)	Load description
1. 1.2 D + 1.6 L	7287			Wt. of tank (full) + Wt. of fdn
2. 0.9 D + 1.0 W	988	123	689	Wt. of tank (empty) + Wt. of fdn + Max (wind)
3. 1.2 D + 1.0 L + 1.0 E	5049	331	934	Wt. of tank (full) + Wt. of fdn. + Max (wind/seis)

(#) Refer to Table 5.3.1: ACI 318–9
D = dead load; L = live load/liquid fill; W = wind load; E = earthquake or seismic load.
Refer to Figure 5.67 for pile group layout

No. of piles = 8 + 8 = 16 $I_{pilegroup} = 6.351\ m^4$ $Z_{pilegroup} = 1.815\ m^4$
$a_2 = a = 0.195$ m

Axial force / pile:

Maximum pile reaction = P / n + [(M/Z) × a]
Minimum pile reaction = P / n − [(M/Z) × a]

Load Case 1 1.2 D + 1.6 LL
Axial compression = 7286 / 16 = 455 kN < 600 kN; safe.

Load Case 2 0.9 D + 1.6 LL
Axial compression = [7287 / 16] + [0.195 × 689 / 1.815]
= 61.76 + 74.02 = 136 kN < 600 kN; safe.
Uplift = 74.02−61.76 = 12 kN (tension occurs) <120 kN; Safe.

Load Case 3 1.2 D + 1.0 L + 1.0 E
Axial compression = [5049 / 16] + [0.195 × 934 / 1.815]
= 316 + 100 = 416 kN < 600 kN; Safe.
Uplift = 316−100 = 216 kN (No uplift)

Check for lateral force:
Maximum lateral force, H = 331 kN No of Piles = 16
Shear per pile = 331 / 16 = 20.69 kN < 50 kN; Safe.

Shear stress in pile cap:

Thickness of Raft $B_{TH} = 0.6$ m

Maximum reaction on a single pile = 455 kN (factored load)
Refer to Figure 5.69 for a critical shear plane diagram around the pile near the edge,
Raft thickness = 0.6M
Edge distance = 0.4 M
Eff. depth, d = 0.475 m (0.6 × 1000 −25 −100) / 1000
Shear area = 1.65 sqm [(0.5 + 0.475) + 2 (0.4 + 0.5 × 0.5 + 0.5 × 0.475)] × 0.6
Shear stress developed = 455 / 1.65 = 276 kN/m^2 = 0.276 N/mm^2 < 0.631 N/mm^2; safe.
Allowable shear stress, Vc = 0.17 λ √fc′= 0.17 × 0.83× √20 = 0.631 N/mm^2
λ = √ [2 / (1+0.004d)] = 0.83 < 1

Flexural strength: Refer to Figure 5.70

Pressure on the pile group

DL + LL (full): P/A	p_1 = 7287 / 52.83 = 137.9 kN/m^2
SL: (+ / −) M / Z	p_2 = 934 / 54.15 = 17.25 kN/m^2
WL: (+ / −) M / Z	p_3 = 689 / 54.15 = 12.72 kN/m^2
Self-weight of the raft:	p_4 = 0.6 × 25 = 15.00 kN/m^2

where A = 52.83 sqm (= A_{BASE}) and Z = 3.14 × 8.2^3 / 32 = 54.15 m^3

Dbase = 8.2 m

Net upward pressure, p_{design}
DL + LL + SL/WL = 137.93 + 17.25−15 = 140 kN/m^2 (Factored)
DL + LL = 137.93 − 15 = 123 kN/m^2 (Factored)
So, p_{design} = 140 kN/m^2

Case 1: When re-bars are placed in radial and circumferential directions

Moment radial (factored):

Span = 4.00 m	Mu_rad = 140 × 4^2 / 10 = 224 kNm	
Strength reduction factor,	ϕ = 0.9 for moment	ACI 318–19 (Table 21.2.1)
	Mu = 224 kNm	tension-controlled
Required nominal strength,	M_n = Mu/ϕ = 249 kNm	ϕ Mn > Mu

Refer to Figure 5.55 for stress-strain diagram
C = T or, 0.85fc'. a. bw = As.fy or, a = As.fy /(0.85 fc' bw)
Let us try with 25 mm diameter bars @ 250 c/c Ast = 1964 mm^2
fc′ = 20 N/mm^2 bw = 1000 mm fy = 450 MPa d = 475 mm
a =1964 × 450 / (0.85 × 20 × 1000) = 52 mm
ϕ Mn = ϕ As fy (d – a/2) = 0.9 × 1964 × 450 × (475 – 52 / 2) / 10^6 =357 kNm >Mu; safe.
Provide bars along the radial direction at top and bottom faces:
25 mm diameter bars @ 250 c/c Ast provided = 1964 mm^2 > Minimum reinforcement; safe

Minimum reinforcement:

As min = 0.0018 Ag = 0.0018 × 1000 × 475 = 855 mm^2

Moment circumferential

Span = 2.75 m Mu_circ = 140 × 2.75^2 / 10 = 87 kNm

Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)

Mu = 87 kNm tension-controlled

Required nominal strength, $M_n = Mu/\phi$ = 97 kNm ϕ Mn > Mu

Refer to Figure 5.55 for a stress-strain diagram

C = T or, 0.85fc'. a. bw = As.fy or, a = As.fy /(0.85 fc' bw)

Let us try with 20 mm diameter bars @ 300 c/c Ast = 1048 mm^2

fc′ = 20 N/mm^2 bw = 1000 mm fy = 450 MPa d = 475 mm

a =1048 × 450 / (0.85 × 20 × 1000) = 28mm

ϕ Mn = ϕ As fy (d – a/2) = 0.9 × 1048 × 450 × (475 – 28 / 2) / 10^6 =197 kNm >Mu; safe.

Provide bars along the circumferential direction at top and bottom faces:

20 mm diameter bars @ 300 c/c Ast provided = 1048 mm^2 > minimum reinforcement; safe

Case 2: Alternate to Case 1, when re-bars are placed in the orthogonal direction

Mortho = [Mu_rad + Mu_circ] / 1.414 = (224 + 87) / 1.414 = 220 kNm

Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)

Mu = 220 kNm tension-controlled

Required nominal strength, $M_n = Mu/\phi$ = 244 kNm

Refer to Figure 5.55 for a stress-strain diagram

C = T or, 0.85fc'. a. bw = As.fy or, a = As.fy /(0.85 fc' bw)

Let us try with 25 mm diameter bars @ 275 c/c Ast = 1786 mm^2

fc′ = 20 N/mm^2 bw = 1000 mm fy = 450 MPa d = 475 mm

a =1786 × 450 / (0.85 × 20 × 1000) = 47mm

ϕ Mn = ϕ As fy (d – a/2) = 0.9 × 1786 × 450 × (475 – 47 / 2) / 10^6 =327 kNm >Mu; safe.

Provide bars along the circumferential direction at top and bottom faces:

25 mm diameter bars @ 275 c/c Ast provided = 1786 mm^2 > minimum reinforcement; safe

For reinforcement arrangement drawing, see Figure 5.71.

5.6 DESIGN OF THE BEAM

5.6.1 Singly reinforced beam ACI 318–19 US Customary units

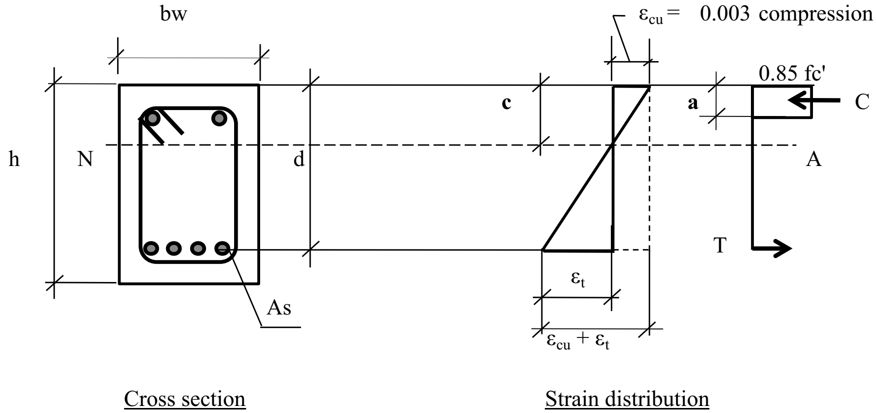

FIGURE 5.52 Stress and strain diagram of a beam section

Design parameters:

fc′ = 3000 psi	Span = 19.68 ft
fy = 60000 psi	bw = 10 inch
Es = 29000000 psi	h = 24 inch > span / 16

Maximum bar diameter = 0.75 inch (#6) cover, d′ = 1.5 inch
Effective depth, d = 24−1.5−0.5 × 0.75 = 22.13 inch

Design load: (Factored Load)

Mu Span = 150 kip ft
Mu support = 190 kip-ft
Vu = 45 kip

Reinforcement:

At mid span

Main bars		*Bar #*	*Bar dia*	*Area (Ast)*
Top	2 nos.	6	(0.75 inch)	0.88 in^2
Bottom	6 nos.	6	(0.75 inch)	2.64 in^2

Stirrups: *2 Legged #3 @ 10 inch c/c (s);* *(Asv = 0.22 in^2 /set)*

At support

Main bars		*Bar #*	*Bar dia*	*Area (Ast)*
Top	6 nos.	6	(0.75 inch)	2.64 in^2
Bottom	2 nos.	6	(0.75 inch)	0.88 in^2

Stirrups: *2 Legged #3 @ 4 inch c/c (s);* *(Asv = 0.22 in^2 /set)*

Strength design:

To find out NA depth

Maximum usable strain at concrete compression fibre, $\varepsilon_{cu} = 0.003$
Net tensile strain at steel reinforcement, $\varepsilon_t = 0.005$
[>= εt y + 0.003] ACI 318–19 (21.2.2.1)
$\varepsilon_{ty} = fy / Es = 0.002$

Tension reinforcement yielded

Depth of the neutral axis from the top, c = εcu. d / (εcu + εt)
c = 0.003 × 22.13 / (0.003 + 0.005) = 8.3 inch.

a = β. c ACI 318–19 (21.2.2.4.1)
β = 0.85 ACI 318–19 (Table 22.2.2.4.3)
a = 0.85 × 8.3 = 7.06 inch

To find out minimum reinforcement, As $_{min}$:

In flexure: As $_{min}$ will be greater of a) and b) ACI 318–19 (9.6.1.2)

a) 3 √fc′. bw. d / fy = 3 × √3000 × 10 × 22.13 / 60000 = 0.61 in^2
b) 200 bw. d / fy = 200 × 10 × 22.13 / 60000 = 0.74 in^2

So, As $_{min}$ = 0.74 in^2

Shear reinforcement: ACI 318–19 (Table 9.6.3.4)
Av $_{min}$ / s, will be greater of a) and b)
where s = spacing of stirrups

a) 0.75 √fc′. bw / fyt = 0.75 × √3000 × 10 / 60000 = (0.01 × s) in^2
b) 50. bw / fyt = 50 × 10 / 60000 = (0.01 × s) in^2

So, Av $_{min}$ = (0.01 × s) in^2

At support

Flexural reinforcement as a singly reinforced beam

Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)
Mu = 190 kip-ft Required nominal strength, M_n = Mu/ϕ = 211 kip-ft
C = 0.85 fc′. a. bw = 0.85 × 3000 × 7.06 × 10 / 1000 = 180 kips
Mn_1 = C (d − a/2) = 180 × (22.13 − 0.5 × 7.06) / 12 = 279 kip-ft > *Mn rqd; singly reinforced.*
As rqd = Mn /fy. (d − a/2) = 211 × 1000 × 12 / 60000 × (22.13 − 0.5 × 7.06) = 2.27 in^2

Tension reinforcement provides at the top, As = 2.64 in^2 > 2.27 in^2 Hence, safe.

Shear reinforcement at support
Strength reduction factor, ϕ = 0.75 for shear ACI 318–19 (Table 21.2.1)

$Vu = 45$ kip Required nominal strength, $V_n = Vu / \phi = 60$ kip

Now, $Vn = Vc + Vs$, where Vc is the shear capacity of the concrete beam section and Vs is the capacity of shear reinforcement in the form of stirrups or bentup bars.
$Vc = 2\ \lambda\ \sqrt{fc'}.\ bw.d = 2 \times 0.79 \times \sqrt{3000} \times 10 \times 22.13 = 19$ kips ACI 318–19 (Table 22.5.5.1)
$\lambda = \sqrt{[2 / (1 + d/10)]} = 0.789 <= 1$ ACI 318–19 (22.5.5.1.3)

Vc < Vn, so provide shear reinforcement
Let us provide, two-legged stirrups #3 @ 4 inch centers ($Av = 0.22\ in^2$ /set)
$Av_{min} = 0.01 \times 4 = 0.04\ in^2$

Now, $Vs = Av.\ fyt.\ d / s = 0.22 \times 60000 \times 22.13 / 4$
$= 73.03$ kips ACI 318–19 (22.5.8.5.3)
$Vn = Vc + Vs = 19 + 73.03 = 92$ kips > Vn rqd; Safe,

Maximum allowable shear in this C/S of the beam, Vu max = 73 kips
$Vu <= \phi\ (Vc + 8.\ \sqrt{fc'}.\ bw.d)$ ACI 318–19 (22.5.1.2)
Vu = 45 kips < Vu max; Okay.

At span

Flexural reinforcement as a singly reinforced beam

Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 150 kip-ft Required nominal strength, $M_n = Mu/\phi = 167$ kip-ft
From Figure 5.72 above, we get
a = 7.06 inch *c = 8.3 inch* $\varepsilon_{cu} = 0$

$C = 0.85\ fc'.\ a.\ bw = 0.85 \times 3000 \times 7.06 \times 10 / 1000 = 180$ kips
$Mn_1 = C\ (d - a/2) = 180 \times (22.13 - 0.5 \times 7.06) / 12 = 279$ kip-ft > *Mn; Singly reinforced.*
As1 rqd $= Mn / fy.\ (d - a/2) = 167 \times 1000 \times 12 / 60000 \times (22.13 - 0.5 \times 7.06) = 1.79\ in^2$

Reinforcement provides at the bottom of the mid span, $As = 2.64\ in^2 > 1.79\ in^2$ Hence, safe.

5.6.2 Singly reinforced beam ACI 318–19 SI units

Design parameters:

fc′ = 20 MPa	Span = 6 m
fy = 415 MPa	bw = 250 mm
Es = 200000 MPa	h = 600 mm > span / 16

Max. bar diameter = 20 mm cover, d′ = 40 mm
Effective depth, $d = 600 - 40 - 0.5 \times 20 = 550$ mm
Design load: (factored load)

Mu Span = 200 kNm
Mu support = 250 kNm
Vu = 200 kN

Reinforcement:

At the mid span

Top layer – 2 nos. 20 mm diameter Ast = 628 mm^2

Bottom layer – 6 nos. 20 mm diameter Ast = 1884 mm^2

Stirrups – 2 legged 8 mm diameter @ 250 mm c/c Asv = 101 mm^2 / set

At support

Top layer – 6 nos. 20 mm diameter Ast = 2198 mm^2

Bottom layer – 2 nos. 20 mm diameter Ast = 628 mm^2

Stirrups – 2 legged 8 mm diameter @ 100 mm c/c Asv = 100.6 mm^2 / set

Strength design:

Refer to Figure 5.72.

To find out NA depth

Maximum usable strain at concrete compression fibre, $\varepsilon_{cu} = 0.003$

Net tensile strain at steel reinforcement, $\varepsilon_t = 0.005$

[>= εt y + 0.003] ACI 318–19 (21.2.2.1)

ε_{ty} = fy / Es = 0.002

Tension reinforcement yielded

Depth of the neutral axis from the top, c = εcu. d / (εcu + εt)

c = 0.003 × 550 / (0.003 + 0.005) = 206 mm

a = β. c ACI 318–19 (21.2.2.4.1)

β = 0.85 ACI 318–19 (Table 22.2.2.4.3)

a = 0.85 × 206 = 175 mm

To find out minimum reinforcement, As $_{min}$:

In flexure: As $_{min}$ will be greater of a) and b) ACI 318–19 (9.6.1.2)

c) 0.25 √fc′. bw. d / fy = 0.25 × √20 × 250 × 550 / 415 = 370 mm^2

d) 1.4 bw. d / fy = 1.4 × 250 × 550 / 415 = 464 mm^2

So, As $_{min}$ = 464 mm^2

Shear reinforcement: ACI 318–19 (Table 9.6.3.4)

Av $_{min}$ / s, will be greater of a) and b)

where s = spacing of stirrups

c) 0.062 √fc′. bw / fyt = 0.062 × √20 × 250 / 415 = (0.17 × s) mm^2

d) 0.35. bw / fyt = 0.35 × 250 / 415 = (0.21 × s) mm^2

So, Av $_{min}$ = (0.21 × s) mm^2

At support

Flexural reinforcement:

Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)

Mu = 250 kNm Required nominal strength, M_n = Mu/ϕ = 278 kNm

$C = 0.85\ fc'.\ a.\ bw = 0.85 \times 20 \times 175 \times 250 / 1000 = 744$ kN
$Mn_1 = C\ (d - a/2) = 744 \times (550 - 0.5 \times 175) / 1000 = 344$ kNm > *Mn rqd; singly reinforced.*

As rqd = Mn /fy. $(d - a/2) = 278 \times 1000000 / 415 \times (550 - 0.5 \times 175) = 1447\ mm^2$

	Required	Provided	
Total tensile reinforcement, As =	$1447\ mm^2$	$1884\ mm^2$	Safe.

Shear reinforcement at support
Strength reduction factor, $\phi = 0.75$ for shear ACI 318–19 (Table 21.2.1)
Vu = 200 kN Required nominal strength, $V_n = Vu / \phi = 267$ kN

Now, Vn = Vc + Vs, where Vc is the shear capacity of the concrete beam section and
Vs is the capacity of shear reinforcement in the form of stirrups or bentup bars.
$Vc = 0.17\ \lambda \sqrt{fc'}.\ bw.d = 0.17 \times 0.79 \times \sqrt{20} \times 250 \times 550 = 83$ kN ACI 318–19 (Table 22.5.5.1)
$\lambda = \sqrt{[2 / (1 + 0.004d)]} = 0.79 <= 1$ ACI 318–19 (22.5.5.1.3)

Vc < Vn, so provide shear reinforcement
Let us provide 2-legged stirrups 8 mm diameter @ 100 mm centers (Asv = $100\ mm^2$ /set)
$Asv_{min} = 0.21 \times 100 = 21\ mm^2$

Now, Vs = Av. fyt. d / s = $100 \times 415 \times 550 / 100$
= 228 kN ACI 318–19 (22.5.8.5.3)
Vn = Vc + Vs = 83 + 228 = 311 kN > Vn rqd; Safe,

Maximum allowable shear in this C/S of beam, Vu max = 304 kN
$Vu <= \phi\ (Vc + 0.66.\ \sqrt{fc'}.\ bw.d)$ ACI 318–19 (22.5.1.2)
Vu = 200 kN < Vu max; Ok.

At span

Flexural reinforcement as a singly reinforced beam

Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 200 kNm Required nominal strength, $M_n = Mu/\phi = 222$ kNm
From the stress block above, we get
a = 175mm *c = 206 mm* $\varepsilon_{cu} = 0$
$C = 0.85\ fc'.\ a.\ bw = 0.85 \times 20 \times 175 \times 250 / 1000 = 744$ kN
$Mn_1 = C\ (d - a/2) = 744 \times (550 - 0.5 \times 175) / 1000 = 344$ kN > *Mn; singly reinforced.*
As_1 rqd = Mn /fy. $(d - a/2) = 222 \times 1000000 / 415 \times (550 - 0.5 \times 175) = 1158\ mm^2$
Ast provided = $1884\ mm^2 > 1158\ mm^2$; Safe.

5.6.3 Doubly reinforced beam – ACI Code

Reference code: ACI 318–19 SI – metric stress in Mpa

Design parameters:

$f_c' = 20$ MPa   Span = 6 m
$f_y = 415$ MPa $b_w = 250$ mm
$E_s = 200000$ MPa $h = 600$ mm > span / 16

Maximum bar diameter = 20 mm   cover, $d' = 40$ mm
Effective depth, $d = 600 - 40 - 0.5 \times 20 = 550$ mm

Design load: (factored load)
Mu Span = 200 kNm
Mu support = 330 kNm
Vu = 200 kN

Reinforcement:

At the mid span

Top layer –	2 nos. 20 mm diameter	Ast = 628 mm^2
Bottom layer –	4 nos. 20 mm diameter	Ast = 1256 mm^2
Stirrups – 2-legged 8 mm diameter @ 300 mm c/c		Asv = 100.6 mm^2 / set

At support

Top layer –	7 nos. 20 mm diameter	Ast = 2198 mm^2
Bottom layer –	2 nos. 20 mm diameter	Ast = 628 mm^2
Stirrups – 2-legged 8 mm diameter @ 100 mm c/c		Asv = 100.6 mm^2 / set

Strength design:

Refer to Figure 5.72.

To find out NA depth

Maximum usable strain at concrete compression fibre, $\varepsilon_{cu} = 0.003$
Net tensile strain at steel reinforcement, $\varepsilon_t = 0.005$
[>= εt y + 0.003]   ACI 318–19 (21.2.2.1)
$\varepsilon_{ty} = f_y / E_s = 0.002$
Tension reinforcement yielded
Depth of the neutral axis from the top, $c = \varepsilon cu.\ d / (\varepsilon cu + \varepsilon t)$
$c = 0.003 \times 550 / (0.003 + 0.005) = 206$ mm

$a = \beta.\ c$ ACI 318–19 (21.2.2.4.1)
$\beta = 0.85$ ACI 318–19 (Table 22.2.2.4.3)
$a = 0.85 \times 206 = 175$ mm

To find out minimum reinforcement, As $_{min}$:
In flexure: As $_{min}$ will be greater of a) and b)   ACI 318–19 (9.6.1.2)

e) $0.25 \sqrt{fc'}.\ bw.\ d / fy = 0.25 \times \sqrt{20} \times 250 \times 550 / 415 = 370\ mm^2$
f) $1.4\ bw.\ d / fy = 1.4 \times 250 \times 550 / 415 = 464\ mm^2$

So, As $_{min}$ = 464 mm^2

Shear reinforcement: ACI 318–19 (Table 9.6.3.4)
Av_{min} / s, will be greater of a) and b)
where s = spacing of stirrups

e) $0.062 \sqrt{fc'}$. bw / fyt = $0.062 \times \sqrt{20} \times 250 / 415 = (0.17 \times s)$ mm^2
f) 0.35. bw / fyt = $0.35 \times 250 / 415 = (0.21 \times s)$ mm^2

So, $Av_{min} = (0.21 \times s)$ mm^2

At support

Flexural reinforcement:

Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 330 kNm Required nominal strength, $M_n = Mu/\phi = 367$ kNm
C = 0.85 fc′. a. bw = $0.85 \times 20 \times 175 \times 250 / 1000 = 744$ kN
Mn_1 = C (d − a/2) = $744 \times (550 - 0.5 \times 175) / 1000 = 344$ kNm < *Mn rqd; doubly reinforced.*

As rqd = Mn /fy. (d − a/2) = $367 \times 1000000 / 415 \times (550 - 0.5 \times 175)$ = 1910 mm^2

For balanced section
The maximum area of reinforcing steel in a singly reinforced section = T / fs
Here, T = C =744 kN
And, fs = fy = 415 MPa As1 = T / fy =744 ×1000 / 415 =1792 mm^2

Provide flexural reinforcement as a doubly reinforced beam.
$Mn_2 = Mn\ rqd - Mn_1 = 367–344 = 23$ *kNm*
$Mn_2 = Cs. (d - d')$ *where Cs = compression in steel at the top and*
d′ = top cover
$Cs = Mn_2 / (d - d') = 23 \times 1000 / (550 - 40) = 44$ *kN*
$\varepsilon_s = \varepsilon_{cu}. (c - d') / c = 0$ *where* ε_s *= strain in comp re-bar at the top > ey, comp steel yielded.*

Now, stress in comp steel, fs' = fy = 415 MPa
After deduction of equivalent concrete stress, Cs = (fs' − 0.85fc′). As_2

$As_2 = Cs / (fs' - 0.85 fc') = 44 \times 1000 / (415–0.85 \times 20) = 112$ *mm*^{2}

As1 + As2 =1910 + 112 = 2022 mm^2

	Required	Provided	
Total compression reinforcement, As_2 =	112 mm^2	628 mm^2	Safe.
Total tensile reinforcement, As_1 =	2022 mm^2	2198 mm^2	Safe.

Shear reinforcement at support
Strength reduction factor, $\phi = 0.75$ for shear ACI 318–19 (Table 21.2.1)

Vu = 200 kN Required nominal strength, V_n = Vu / ϕ = 267 kN

Now, Vn = Vc + Vs, where Vc is the shear capacity of the concrete beam section and

Vs is the capacity of shear reinforcement in the form of stirrups or bentup bars.

Vc = 0.17 λ $\sqrt{fc'}$. bw.d = 0.17× 0.79 × $\sqrt{20}$ × 250 × 550 = 83 kN ACI 318–19 (Table 22.5.5.1)

$\lambda = \sqrt{[2 / (1 + d/10)]}$ = 0.79 <=1 ACI 318–19 (22.5.5.1.3)

Vc < Vn, so provide shear reinforcement

Let us provide, 2-legged stirrups 8 mm diameter @ 100 mm centers (Asv = 100 mm^2 /set)

Asv $_{min}$ = 0.21 × 100 = 21 mm^2

Now, Vs = Av. fyt. d / s = 100 × 415 × 550 / 100
= 228 kN ACI 318–19 (22.5.8.5.3)

Vn = Vc + Vs = 83 + 228 = 311 kN > Vn rqd; Safe,

Maximum allowable shear in this C/S of the beam, Vu max = 307 kips

Vu <= ϕ (Vc + 0.66. $\sqrt{fc'}$. bw.d) ACI 318–19 (22.5.1.2)

Vu = 200 kN < Vu max; Ok.

At Span

Flexural reinforcement as a singly reinforced beam

Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)

Mu = 200 kNm Required nominal strength, M_n = Mu/ϕ = 222 kNm

From Figure 5.72 above, we get

a = 175mm *c = 206 mm* $\varepsilon_{cu} = 0$

C = 0.85 fc′. a. bw = 0.85 × 20 × 175× 250 / 1000 = 744 kN

Mn_1 = C (d − a/2) = 744 × (550 − 0.5 × 175) / 1000 = 344 kN > *Mn; singly reinforced.*

As_1 rqd = Mn /fy. (d − a/2) = 222 × 1000000 / 415 × (550 − 0.5 × 175) = 1158 mm^2

Ast provided = 1256 mm^2 > 1158 mm^2; Safe.

5.6.4 Singly reinforced beam – limit state method (IS 456: 2000 & SP 16)

Design parameters:

Beam size: 250 x 450

B = 0.25 m

D = 0.45 m

Cover = 40 mm

Span = 6 m

fck = 20 MPa

fy = 500 MPa

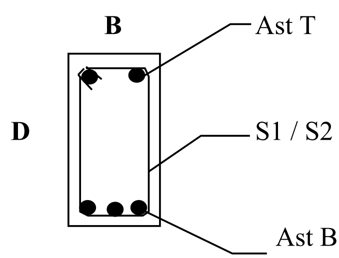

FIGURE 5.53 Singly reinforced beam

Design load

Mspan = 40 kNm Mu span = 48 kNM (factored)

Msupport = 50 kNm   Mu span = 60 kNM   (factored)
F = 20 kN   Fu = 24 kN   (factored)

Reinforcement:

At mid span

Top layer – 2 nos. 16 mm diameter Ast = 402 mm^2
Bottom layer – 2 nos. 16 mm diameter Ast = 402 mm^2

Stirrups – 2-legged 8 mm diameter @ 300 c/c

At support

Top layer – 2 nos. 16 mm diameter Ast = 402 mm^2
Bottom layer – 2 nos. 16 mm diameter Ast = 402 mm^2

Stirrups, S1 – 2-legged 8 mm diameter @ 200 c/c

Design check:

Flexural reinforcement:

Mid span

Effective depth, d = 450−40−0.5 × 16 = 402 mm
M_U limit/bd^2 = 2.66 [Table D – SP 16]
M_U limit = 2.66 × 250 × 402^2 / 10^6 = 107 kNm
Span moment = 48 kNm < M_U limit   singly reinforced beam

M_u /bd^2 = (48 × 10^6) / (250 × 402^2) = 1.19
Reinforcement at the bottom layer: [Table 2 – SP 16]
pt = 0.298 %   Percentage of steel required for design moment
Ast required = 0.298 × 250 × 402 / 100 = 299 mm^2/m width
Ast provided = 402 mm^2/m width   > Ast reqd; safe.
Ast min = (0.85 / fy) b. d = (0.85 /500) × 250 × 402 = 171 mm^2 [IS 456: 2000–26.5.1.1]

Reinforcement at the top layer:
Provide minimum reinforcement.
Ast reqd. = 171 mm^2/m width
Ast provided = 402 mm^2/m width;   Okay.

End support

Support moment = 60 kNm   < M_U limit   Singly reinforced beam

dprov = 402 mm
M_u /bd^2 = (60 × 10^6) / (250 × 402^2) = 1.49
Flexural reinforcement at the top layer: [Table 2 – SP 16]
pt = 0.382 %   percentage of steel required for design moment
Ast required = 0.382 × 250 × 402 / 100 = 384 mm^2/m width
Ast provided = 402 mm^2/m width   > Ast reqd; safe.
Ast min = = 171 mm^2

Flexural reinforcement at the bottom layer:
Ast min = 171 mm^2/m width
Ast provided = 402 mm^2/m width > Ast min; safe.

Shear strength:

End shear, Fu = 24 kN
Percentage of flexural reinforcement, pt = 0.382 %
Permissible shear stress, τ_c = 0.39 N/mm^2 [IS 456: 2000 – Table 19]
Permissible shear strength, Vu = τc.b.d = 0.39 × 250 × 402 / 1000 = 39 kN >Fu; safe.
Provide nominal shear reinforcement, S2 = 2 Legged – 8 mm diameter @ 300 c/c.

5.6.5 Doubly reinforced beam – limit state method (IS 456: 2000 & SP 16)

Design parameters:

Beam size: 300 x 600
B = 0.30 m
D = 0.60 m
Cover = 40 mm
Span = 7.5 m
fck = 20 MPa
fy = 500 MPa

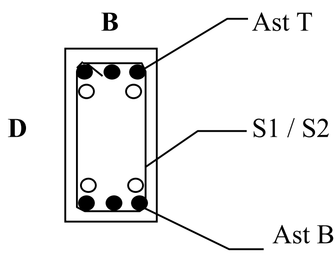

FIGURE 5.54 Doubly reinforced beam

Design load

Mspan = 360 kNm	Mu span = 540 kNM	(factored)
Msupport = 301 kNm	Mu span = 452 kNM	(factored)
Fsupp = 240 kN	Fu supp = 360 kN	(factored)

Reinforcement:

At Mid span

Top layer –	3 nos.	28 mm diameter	Ast = 1846 mm^2
Bottom layer –	5 nos.	28 mm diameter	Ast = 3077 mm^2
Stirrups, S2 –	2-legged	10 mm diameter @ 300 c/c	

At End supports

Top layer –	5 nos.	28 mm diameter	Ast = 3077 mm^2
Bottom layer –	3 nos.	28 mm diameter	Ast = 1846 mm^2
Stirrups, S1 –	2-legged	10 mm diameter @ 150 c/c	

Design check:

Flexural reinforcement at span:
Effective depth, d = 600−40−0.5 × 28 = 546 mm
M_U limit/bd^2 = 2.76 [Table D – SP 16]
M_U limit = 2.76 × 300 × 546^2 / 10^6 = 247 kNm
Span moment = 540 kNm > M_U limit Doubly reinforced beam

$M_u /bd^2 = (540 \times 10^6) / (300 \times 546^2) = 6.04$

At the bottom layer:
Cover / d = 40 / 546 = 0.07 ~ 0.10
pt = 0.163 % steel in tension [Table 54 – SP 16]
Ast required = 1.63 × 300 × 546 / 100 = 2670 mm^2/m width
Ast provided = 3077mm^2/m width > Ast reqd; safe.
Ast min = (0.85 / fy) b. d = (0.85 /500) × 300 × 600 = 306 mm^2

At the top layer:
pt = 0.95 % steel in compression [Table 54 – SP 16]
Ast required = 0.95 × 300 × 546 / 100 = 1554 mm^2/m width
Ast provided = 1846 mm^2/m width > Ast reqd; Safe.

Flexural reinforcement at support:

Support moment = 452 kNm > M_U limit doubly reinforced beam
M_U limit = 247 kNm
dprov = 546 mm
$M_u /bd^2 = (452 \times 10^6) / (300 \times 546^2) = 5.05$

At the top layer:
pt = 1.38 % Steel in tension [Table 54 – SP 16]
Ast required = 1.38 × 1000 × 0.3 × 546 / 100 = 2260 mm^2/m width
Ast provided = 3077 mm^2/m width > Ast reqd; safe.
Ast min = 306 mm^2/m width

At the bottom layer:
pc = 0.67 % Steel in compression [Table 54 – SP 16]
Ast required = 0.67 × 1000 × 0.3 × 546 / 100 = 1097 mm^2/m width
Ast provided = 1846 mm^2/m width > Ast reqd; Safe.
Ast min = 306 mm^2/m width

Shear strength:

End shear, Fu support = 360 kN
Percentage of flexural reinforcement provided at support, pt
=100 Ast / b. d = 100 × 3077 / (300 × 546) = 1.88 %
Permissible shear stress, τ_c = 0.76 N/mm^2 [Table 61 – SP 16]
Shear capacity of the beam section, Vu = τc.b.d = 0.76 × 300 × 546 / 1000 = 124 kN
< Fu support; provide shear reinforcement.

Shear reinforcement provided:

Stirrups S1: 2 legged – 10 diameter @ 150 mm c/c Asv = 158 mm^2 / set
Shear capacity of stirrups, Vus = 0.87. fy. Asv.d / sv
Vus = (0.87 × 500 × 158 × 546 / 150) / 1000 = 250 kN
Total shear capacity = τc.b.d + Vus = 124 + 250 = 374 kN > 360 kN; safe.

5.7 DESIGN OF COLUMN

5.7.1 Design of column – ACI Code method SI units

Design parameters:

Concrete grade = M25 fc′ = 25 N/mm^2 fy = 415 N/mm^2
Ec = 5700 √25 = 28500 N/mm^2

TABLE 5.59
Column dimensions

Column Mkd	Size		Unsupported length		Slenderness ratio	
	D (h)	B (b)	lex	ley	lex /D	ley/B
	M	m	m	m		
C1	0.6	0.4	6	3	10.00 Not slender	7.50 Not slender

Clear cover = 40 mm Ag = 240000 mm^2 d’ = 52.5 mm γ = 0.8

Main reinforcement

As1 = 3–25 diameter per side
As2 = 3–25 diameter per side
Ag = 240000 mm^2
Ast = As1 + As2 = 5888 mm^2
p = 100 × 5888 / 240000 = 2.5 %
Minimum reinforcement = 0.01 Ag = 2400 mm^2
Maximum reinforcement = 0.08Ag =19200 mm^2
ACI 318–19 (10.6.1.1)

Design load (factored):

Pu = 2500 kN Mux = 400 kNm Muy = 50 kNm

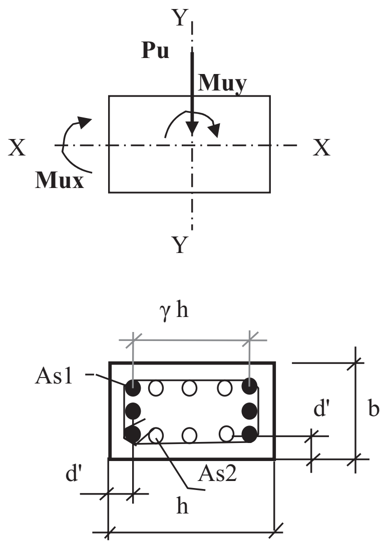

FIGURE 5.55 Column section

Check for slenderness effect:

lex / D = 6000 / 600 = 10
ley / B = 3000 / 7500 =7.5 < 22; short column; slenderness effect is not to be considered.

[How to check the slenderness effect?
a. Slenderness effect (klu/r) is neglected if, lex/D or ley/B <= 22 ACI 318–19 (6.2.5)
b. If, Slenderness ratio limit exceeds, moments value to be magnified as below:

Critical buckling load, $Pc = \pi^2 (EI)_{eff} / (k.lu)^2 = 14050\ kN$ *ACI 318–19 (6.6.4.4.2)*

About the major axis:

$EI\ eff = 0.4.\ Ec.\ Ig / (1+ \beta_{dns}) = 0.25\ Ec\ Ig$, *where* $(1+ \beta_{dns}) = 0.6$
$= 0.25 \times 28500 \times (400 \times 600^3 / 12)$
$= 51.3 \times 10^{12}$
$(klu)^2 = 6000^2 = 36 \times 10^6$ *for* $k = 1$
$Pc = 3.14^2 \times 51.3 \times 10^{12} / 36 \times 10^6 / 1000 = 14050\ kN$

About the minor axis:
$EI\ eff = 0.4.\ Ec.\ Ig / (1+ \beta_{dns}) = 0.25\ Ec\ Ig$, *where* $(1+ \beta_{dns}) = 0.6$
$= 0.25 \times 28500 \times (600 \times 400^3 / 12)$
$= 22.8 \times 10^{12}$
$(klu)^2 = 3000^2 = 9 \times 10^6$ *for* $k = 1$
$Pc = 3.14^2 \times 22.8 \times 10^{12} / 9 \times 10^6 / 1000 = 24978\ kN$

Moment magnification factor, $\lambda = Cm / (1 - Pu/0.75Pc) >= 1$
$Cm = 1$ $\lambda = 1 / (1–2500 / 0.75 \times 14050) = 1.31$
Amplified moment for design, $Mc = \lambda\ M_2$, *where* M_2 *is first-order factored moment]*
In this example, the column section is not slender about major or minor axis, so the magnification factor is not used.

Capacity checks by PCA load contour method:

Pu = 2500 kN Pn = Pu/ ϕ = 3846 kN
Mux = 400 kNm Mnx = Mux / ϕ = 615 kNm
Muy = 50 kNm Mny = Muy / ϕ = 77 kNm
ϕ = strength reduction factor = 0.65 ACI 318–19 (Table 21.2.1&2)
P_0 = nominal axial strength at zero eccentricity ACI 318–19 (Table 22.4.2.2)
= 0.85 fc′ (Ag – Ast) + fy Ast
= 0.85 × 25 × (240000–5888) + 415 × 5888 = 7418 kN
Pu max = 0.8 P_0 = 0.8 × 7418 = 5934 kN > Pn; Safe.
Mnox and Mnoy = equivalent uniaxial bending moment strength
Assume β = 0.65 PCA. Refer to Chapter 7 (Eqn 20)
h = 600 mm
b = 400 mm
Mnox = Mnx + Mny. h/b. ((1–β) / β)
= 615 + 77 × (600 / 400) × (1 – 0.65) / 0.65
= 678 kNm
Mnoy = Mny + Mnx. b/h. ((1–β) / β)
= 77 + 615 × (400 / 600) × (1 – 0.65) / 0.65
= 298 kNm

For, Mny/Mnx >= Mnoy /Mnox, Equivalent uniaxial bending moment = Mnoy
Mny/Mnx <= Mnoy /Mnox, Equivalent uniaxial bending moment = Mnox

Equivalent BM, Mnox = 678 kNm

To find out reinforcement (ρ) required as per interaction diagram (R3 – 60.8) in SP-17 (14) Vol 3

Pn = 3846 kN　　$\gamma = 0.8$
Mnox = 678 kNm (= Pn.e)　　fc′ = 25 N/mm^2 (3.6 ksi)
Ag = 240000 mm^2　　fy = 415 N/mm^2 (60 ksi)　　h = 600 mm

Kn = Pn / fc′. Ag = 3846154 / (25 × 240000) = 0.64
Rn = Pn.e / fc′. Ag. h = 677514793 / (25 × 240000 × 600) = 0.19
Required % of reinforcement, ρ from chart = 0.023
ρ provided = 0.025 > ρ required; hence, safe.

Check for as per the load contour equation, PCA note Chapter 7 (Eqn 14)

$$\left[\frac{Mnx}{Mnox}\right]^{\left[\frac{\log 0.5}{\log\beta}\right]} + \left[\frac{Mny}{Mnoy}\right]^{\left[\frac{\log 0.5}{\log\beta}\right]}$$

$$= \left[\frac{615}{678}\right]^{\left[\frac{\log 0.5}{\log 0.65}\right]} + \left[\frac{77}{298}\right]^{\left[\frac{\log 0.5}{\log 0.65}\right]}$$

= 0.86 + 0.11　　where, log 0.5 / log 0.65 = 1.61
= 0.97 < 1　　Hence, safe.

5.7.2 Design of column – IS Code method

Concrete grade = M20　　fc′ = 20 N/mm^2　　fy = 415 N/mm^2

TABLE 5.60
Column dimensions

Column Mkd	Size D (h)	Size B (b)	Unsupported length lex	Unsupported length ley	Slenderness ratio lex /D	Slenderness ratio ley/B
	M	m	m	m		
C1	0.6	0.4	6	3	10.00 Not slender	7.50 Not slender

Main reinforcement

As1 = 4–25 diameter per face
As2 = 3–25 diameter per face
Ac = 233131 mm^2
As = As1 + As2 = 6869 mm^2
p = 100 × 6869 / 233131 = 2.9 %
Minimum reinforcement = 0.008 Ac = 1865 mm^2
Maximum reinforcement = 0.06Ag = 13988 mm^2

Clear cover = 40 mm d’ = 52.5 mm
Puz = 0.45 fck Ac + 0.75 fy As = 4236 Kn

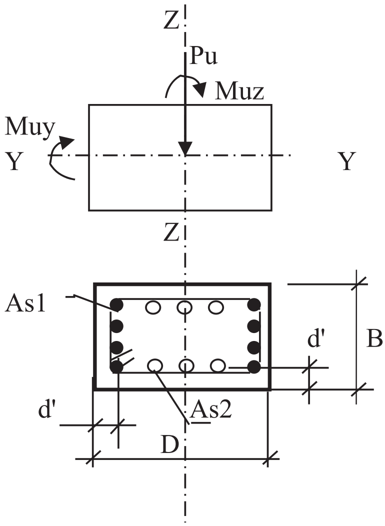

FIGURE 5.56 Column section

TABLE 5.61
Design load for columns (factored load)

SL No	Load case no. and combinations	Axial Pu kN	Moment Muz kNm	Moment Muy kNm
1	12 1.5DL+1.5SLX	2206	239	–12
2	15 1.2(DL+LL–SLX)	265	–132	–8
3	11 1.5DL+1.5LL	601	154	–1
4	11 1.5DL+1.5LL	530	–145	10
5	14 1.5DL–1.5SLZ	1947	11	–128
6	13 1.5DL+1.5SLZ	1812	–12	126
7	14 1.5DL–1.5SLZ	971	–85	–132
8	13 1.5DL+1.5SLZ	1205	–91	111
9	13 1.5DL+1.5SLZ	1256	–13	168
10	14 1.5DL–1.5SLZ	1294	–16	–171
11	12 1.5DL+1.5SLX	1905	283	–9
12	12 1.5DL+1.5SLX	1860	–282	4

TABLE 5.62
Design results – compression member subject to biaxial bending

			About major axis			From chart 32(*)			From chart 32(*)			
L/C	Pu/ Puz	αn	d′/D	p/ f_{ck}	P_u / $f_{ck}bD$	M_{uz}/ $f_{ck}BD^2$	M_{uz}	d′/B	M_{uy}/ $f_{ck}DB^2$	M_{uy}	Stress interaction factor	Remarks
1	0.01	0.682	0.1	0.14	0.46	0.17	490	0.1	0.17	326	0.72	Safe
2	0.00	0.669	0.1	0.14	0.06	0.22	634	0.1	0.22	422	0.42	Safe
3	0.00	0.671	0.1	0.14	0.13	0.24	691	0.1	0.24	461	0.38	Safe
4	0.00	0.670	0.1	0.14	0.11	0.24	691	0.1	0.24	461	0.43	Safe
5	0.01	0.681	0.1	0.14	0.41	0.19	547	0.1	0.19	365	0.56	Safe
6	0.01	0.680	0.1	0.14	0.38	0.2	576	0.1	0.2	384	0.54	Safe
7	0.00	0.674	0.1	0.14	0.20	0.24	691	0.1	0.24	461	0.68	Safe
8	0.01	0.675	0.1	0.14	0.25	0.23	662	0.1	0.23	442	0.66	Safe
9	0.01	0.676	0.1	0.14	0.26	0.23	662	0.1	0.23	442	0.59	Safe
10	0.01	0.676	0.1	0.14	0.27	0.23	662	0.1	0.23	442	0.61	Safe
11	0.01	0.680	0.1	0.14	0.40	0.19	547	0.1	0.19	365	0.72	Safe
12	0.01	0.680	0.1	0.14	0.39	0.2	576	0.1	0.2	384	0.66	Safe

(*) SP 16 – Design Handbook

Stress interaction factor:

$(Mux/Mux1)^{\alpha n} + (Muy/Muy1)^{\alpha n} <= 1$

Or

$(Mz/Muz)^{\alpha n} + (My/Muy)^{\alpha n} <= 1$

5.8 DESIGN OF THE SLAB

Design parameters:

Grade of concrete: M20 fck = 20 N/mm²
Reinforcement bar HYSD fy = 415 N/mm²
(High-yield strength deformed bars)
Unit weight of concrete γ conc = 25 kN/m³

Design load intensity:

Roof slab

	Dead load (DL)	Live load (LL)
120 mm thick slab	3.00 kN/m²	
Roof treatment	1.00 kN/m²	
Plaster at soffit	0.24 kN/m²	1.5 kN/m²
Sum total =	4.24 kN/m²	1.5 kN/m²

Floor slab

	Dead load (DL)	Live load (LL)
125 mm thick slab	3.13 kN/m^2	
Floor finishes	0.80 kN/m^2	
Plaster at soffit	0.24 kN/m^2	
Partition wall	1.00 kN/m^2	5 kN/m^2
Sum total =	5.17 kN/m^2	5 kN/m^2

As per the IS Code method *(IS 456: 2000 and SP16)*

Roof slab

UDL 5.74 kN/m^2 (DL + LL = 4.24 + 1.5)
Span = 2 m

Moment span = $5.74 \times 2^2 / 10 = 2.3$ kNm
Moment span = $5.74 \times 2^2 / 12 = 1.91$ kNm
R1 = 5.74 kN R2 = 5.74 kN
Try with 120 mm thick slab
Width, B = 1000 mm Depth, D = 120 mm cover = 15 mm
fck = 20 MPa fy = 415 MPa

Reinforcement bars:
Span: 8 mm diameter @ 150 mm c/c at the bottom (Ast = 335 mm^2 / m)
Support: 8 mm diameter @ 150 mm c/c at the bottom (Ast = 335 mm^2 / m)

Design check:

Span moment
$Mu_{span} = 1.5 \times 2.3 = 3.45$ kNm Load factor = 1.5
d prov = $120 - 15 - 0.5 \times 8 = 101$ mm
M_U Limit = 28 kNm Singly reinforced.
M_U Limit/bd^2 = 2.76
$M_{u\ span}/bd^2 = 3450000 / (1000 \times 101^2) = 0.34$
pt = 0.12 % minimum steel (SP 16 Table 2)
Ast reqd. = $0.12 \times 1000 \times 120 / 100 = 144$ mm^2/m width
Ast provided = 335 mm^2/m width at the bottom layer > Ast required; safe
Ast min = 144 mm^2/m width

Support moment
$Mu_{support} = 1.5 \times 1.91 = 2.87$ kNm Load factor = 1.5
d prov = 101 mm
M_U Limit = 28 kNm Singly reinforced.
$Mu_{support}/bd^2 = 0.28$
pt = 0.12 % minimum steel (SP 16 Table 2)

Ast reqd. = 0.12 × 1000 × 120 / 100 = 144 mm^2/m width
Ast provided = 335 mm^2/m width at the bottom layer > Ast required; safe
Ast min = 144 mm^2/m width

End shear = 8.61 kN
Allowable shear stress, τc = 0.2 N/mm^2
τc.b.d = 0.2 × 1000 × 101 / 1000 = 20.2 N > End shear; Safe
(Shear check is usually required for slab with openings)

Floor slab

UDL 10.17 kN/m^2 (DL + LL = 5.17 + 5)
Span = 2 m

Moment span = 10.7 × 2^2 / 10 = 4.07 kNm
Moment span = 10.17 × 2^2 / 12 = 3.39 kNm
R1 = 10.17 kN R2 = 10.17 kN
Try with 125 mm thick slab
Width, B = 1000 mm Depth, D = 125 mm cover = 15 mm
fck = 20 MPa fy = 415 MPa

Reinforcement bars:
Span: 8 mm diameter @ 150 mm c/c at bottom (Ast = 335 mm^2 / m)
Support: 8 mm diameter @ 150 mm c/c at bottom (Ast = 335 mm^2 / m)

Design check:
Span moment
Mu_{span} = 1.5 × 4.07 = 6.11 kNm Load factor = 1.5
D prov = 125–15 – 0.5 × 8 = 106 mm
M_U limit = 31 kNm Singly reinforced.
M_U limit/bd^2= 2.76
$M_{u\ span}$ /bd^2 = 6.11 / (1000 × 106^2) = 0.54
pt = 0.16 % minimum steel (SP 16 Table 2)
Ast reqd. = 0.16 × 1000 × 125 / 100 = 198 mm^2/m width
Ast provided = 335 mm^2/m width at the bottom layer > Ast required; safe
Ast min = 150 mm^2/m width (0.12 %)

Support moment
$Mu_{support}$ = 1.5 × 3.39 = 5.09 kNm Load factor = 1.5
D prov = 125–15 – 0.5 × 8 = 106 mm
M_U Limit = 31 kNm Singly reinforced.
$Mu_{support}$ /bd^2 = 0.45
pt = 0.14 % minimum steel (SP 16 Table 2)
Ast reqd. = 0.14 × 1000 × 125 / 100 = 179 mm^2/m width
Ast provided = 335 mm^2/m width at the bottom layer > Ast required; safe
Ast min = 150 mm^2/m width

End shear = 15.26 kN

Allowable shear stress, $\tau c = 0.2$ N/mm^2
$\tau c.b.d = 0.2 \times 1000 \times 106 / 1000 = 21.2$ N > End shear; safe
(Shear check is usually required for slab with openings)

As per ACI Code method *ACI 318–19 SI units*

Floor slab

Span = 2.0 m UDL = 5.17 kN/m^2 DL and 5.0 kN/m^2 LL
Load combination = 1.2 DL + 1.6 LL
Design UDL = $1.2 \times 5.17 + 1.6 \times 5 = 14.2$ kN/m^2

Moment span =$14.2 \times 2^2 /10 = 5.68$ kNm
Moment span =$14.2 \times 2^2 /12 = 4.73$ kNm

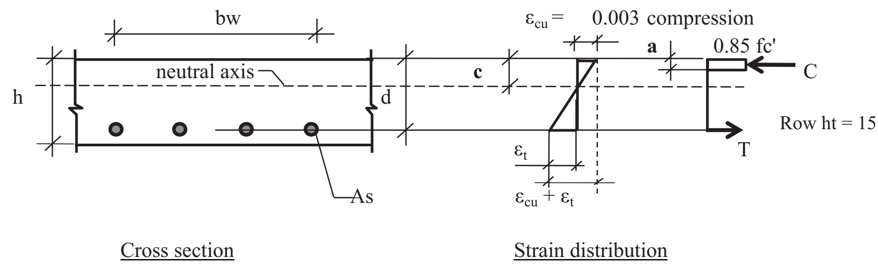

FIGURE 5.57 Stress block diagram in the slab

From the above stress block, $C = T$
Or, $0.85fc'. a. bw = As.fy$
Or, $a = As.fy /(0.85 fc' bw)$
Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)

Span reinforcement:
Mu = 5.68 kNm
Strength reduction factor, $\phi = 0.9$ for Moment ACI 318–19 (Table 21.2.1)
Required nominal strength, $M_n = Mu / \phi = 6.31$ kNm
$fc' = 16$ N/mm^2 fy = 415 MPa
bw = 1000 m D = 125 mm d = 106 mm
Let us try with 8 mm diameter bars @ 150 mm c/c. Ast = 335 mm^2 > min re-bars.

Ast min = $0.18 \times 1000 \times 106 / 100 = 191$ mm^2 / m width.

$a = 335 \times 415 / (0.85 \times 16 \times 1000) = 10$ mm
$\phi Mn = \phi As fy (d - a/2) = 0.9 \times 335 \times 415 \times (106-10/2)/10^6 = 13$ kNm > *Mu; safe.*
Provide reinforcement at top and bottom faces: 8 mm diameter bars @ 150 mm c/c.

Support reinforcement:
Moment support = 4.73 kNm

Mu = 4.73 kNm
Strength reduction factor, ϕ = 0.9 for Moment ACI 318–19 (Table 21.2.1)
Required nominal strength, M_n = Mu / ϕ = 5.26 kNm
fc′ = 16 N/mm² fy = 415 MPa
bw = 1000 m D = 125 mm d = 106 mm
Let us try with 8 mm diameter bars @ 150 mm c/c. Ast = 335 mm² > min re-bars.

Ast min = 191 mm² / m width.

a = 335 × 415 / (0.85 × 16 × 1000) = 10 mm
ϕ *Mn* =ϕ *As fy (d − a/2) = 0.9 × 335 × 415 × (106–10 / 2) / 10⁶ = 13 kNm > Mu; safe.*
Provide reinforcement at top and bottom faces: 8 mm diameter bars @ 150 mm c/c.

For reinforcement arrangement in the slab, see drawing details in Chapter 6

5.9 DESIGN OF EARTH RETAINING WALL

The workout example is a cantilever type earth retaining wall. The cantilever wall is popularly used to protect embankment for road and railway, wing wall for culvert and elevated plot in the plant.

SKETCH

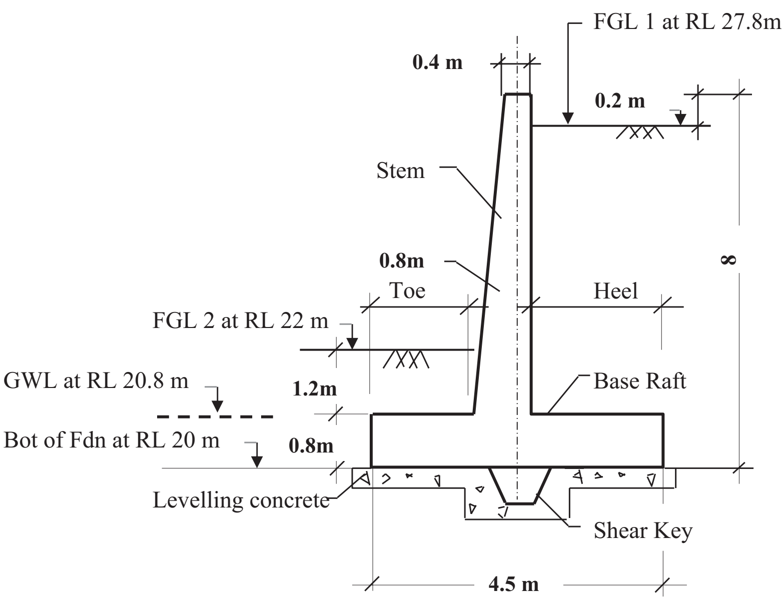

FIGURE 5.58 Detail of retaining wall

Design parameters:

Top of stem wall, TOP = RL 28 m
Finish-grade level, FGL1 = RL 27.8 m
Finish-grade level, FGL2 = RL 22 m

Groundwater level, GWL = RL 20.8 m
Bottom of foundation, BOF = RL 20 m

(RL means plant reference level)

Ground condition and soil parameters:
Gross bearing pressure at the founding level, p_{gross} = 250 kN/m^2
Backfill soil density, γ_{fill} = 18 kN/m^3
Angle of internal friction, ϕ_{fill} = 34 degree
Cohesion, c = 0 kg/cm^2
Materials
Concrete grade: M25 f_{ck} = 25 N/mm^2 Density, γ_{conc} = 25 kN/m^3
Reinforcement steel: Fe415 f_s = 415 N/mm^2
Clear cover = 50 mm Max diameter of re-bar, ϕ_{dia} = 20 mm

Loading:

Ground surcharge: γ_{surch1} = 10 kN/m^2 γ_{surch2} = 10 kN/m^2

Dimensioning:

Height, H = 8 m Base width, B = 4.5 m Base thickness, B_T= 0.8 m
Toe projection, toe = 1.5 m (0.33 × B)
Stem thickness at the top, S_{top} = 0.4 m Stem thickness at the bottom, S_{bot} = 0.8m
Heel = 2.2 m

Earth pressure

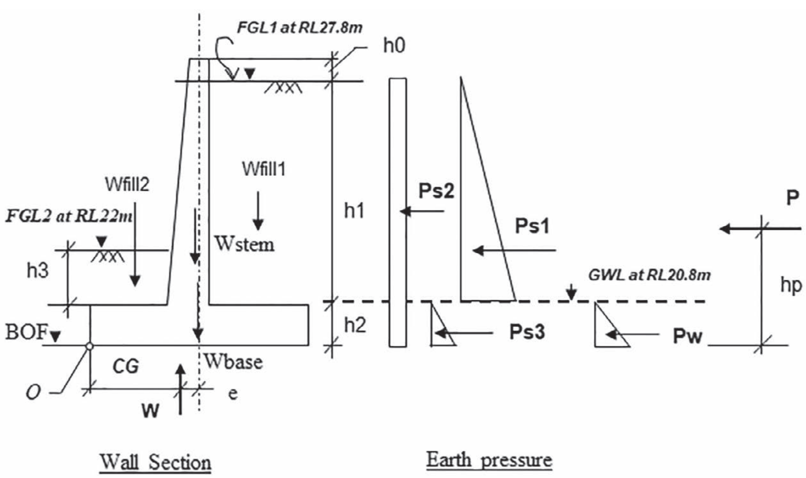

FIGURE 5.59 Earth pressure on the retaining wall

The horizontal forces from backfill earth and groundwater have been described as follows:

Ps_1 = pressure from soil above GWL.
Ps_2 = effective pressure from ground surcharge
Ps_3 = pressure from soil below GWL.
Pw = pressure from GWL.
$P = \Sigma\ (Ps_1 + Ps_2 + Ps_3 + Pw)$

Stem ht. above FGL, $h_0 = 28–27.8 = 0.2$ m
Height of fill above GWL, $h_1 = 27.8–20.8 = 7$ m
Submerged depth of fill, $h_2 = 20.8–20 = 0.8$ m
Depth of fill, $h_3 = 22–20–0.8 = 1.2$ m

Submerged unit wt. of Soil = $\gamma_{sub} = 18 - 10 = 8$ kN/m^3

Coefficient of earth pressure:
Active pressure, ka = 0.28 (1−Sin ϕfill) / (1+Sin ϕfill)
Passive pressure, kp = 3.54 (1+Sin ϕfill) / (1−Sin ϕfill)
At rest, $k_0 = 0.5$

$Ps_1 = 0.5 \times 18 \times 7^2 \times 0.28 = 123.5$ kN
$Ps_2 = 10 \times 0.28 \times (7 + 0.8) = 21.84$ kN
$Ps_3 = 0.5 \times 8 \times 0.8^2 \times 0.28 = 0.72$ kN
$Pw = 0.5 \times 10 \times 0.8^2 = 3.20$ kN
$P = \Sigma\ (Ps_1 + Ps_2 + Ps_3 + Pw) = 149$ kN
Taking the sum of moments about O,
$\Sigma M = 0$, we get
$hp = [(123.5 \times (0.8 + 7/3) + 21.84 \times (7 + 0.8) / 2 + 0.72 \times 0.8 / 3 + 3.2 \times 0.8 / 3)] / 149$
$= 3.18$ m

Weights of backfill soil, surcharge and RCC retaining wall
Wfill1 = $2.2 \times (7 \times 18 + 10) = 299$ kN
Wfill2 = $1.5 \times (1.2 \times 18 + 10) = 47$ kN
Wstem = $0.5 \times (0.4 + 0.8) \times (0.2 + 7) \times 25 = 108$ kN
Wbase = $4.5 \times 0.8 \times 25 = 90$ kN
Total gravity load, W = 299 + 47 + 108 + 90 = 544 kN

Reactions:
Vertical upward: W = 544 kN
Lateral (active pressure): P = 149 kN
Location of resultant soil pressure W = CG = 1.81 m

To find out CG distance:
Let us summarize moments of forces about the point O at the bottom corner of the base, refer to Figure 5.80.

Overturning moment:

– (P x hp) = – (149 × 3.18) = (–) 474 kNm

Restoring moment:

i) [+ wfill1 × Heel × 0.5 + Sbot + Toe] = 299 × [(2.2 × 0.5) + 0.8 + 1.5] = 1017 kNm

ii) [+ wfill2 × 0.5 × Toe] = 47.4 × 0.5 × 1.5 = 36 kNm

iii) [+Wstem × (Toe + 0.5 × Sbot)] = 108 × (1.5 + 0.5 × 0.8) = 205 kNm

iv) [+ Wbase × 0.5B] = 90 × 0.5 × 4.5 = 203 kN

Total restoring moment = 1017 + 36 + 205 + 203 = 986 kNm

Σ M about O = 1017–474 = 986 kNm

Distance of center of gravity of loads from point *O*, CG = Σ M / Rv = 988 / 544 = 1.81 m

[Note: Effect of passive resistance by Fill2 is neglected.]

Stability check

Overturning

Factor of safety (ovr) = W × CG / (P × hp) = 544 × 1.81 / (149 × 3.18) = 2.08 > 1.5; safe.

Minimum safety factor = 1.5

[FOS against overturning should not be less than 1.5 for granular backfill and 2 for cohesive backfill].

Sliding

Factor of safety (slide) = μ W / Rh = 0.55 × 544 / 149 = 2.01 > 1.5 Okay; shear key not required.

Coefficients of friction, μ = 0.55 Minimum safety factor = 1.5

[FOS against sliding should not be less than 1.5; provide shear key when FOS is lower than 1.5]

Note: Effect of passive resistance neglected.

Recommended coefficients of friction, μ (Foundation design – By Wayne.C.Teng)
Course-grained soils (without silt) – 0.55
Course-grained soils (without silt) – 0.45
Silt – 0.35
Sound rock (with rough surface) – 0.6

Strength design of RCC base raft and wall

According to IS 456 x: 2000

Method of RCC design: Limit state Load factor = 1.5

Calculated gross bearing pressure:

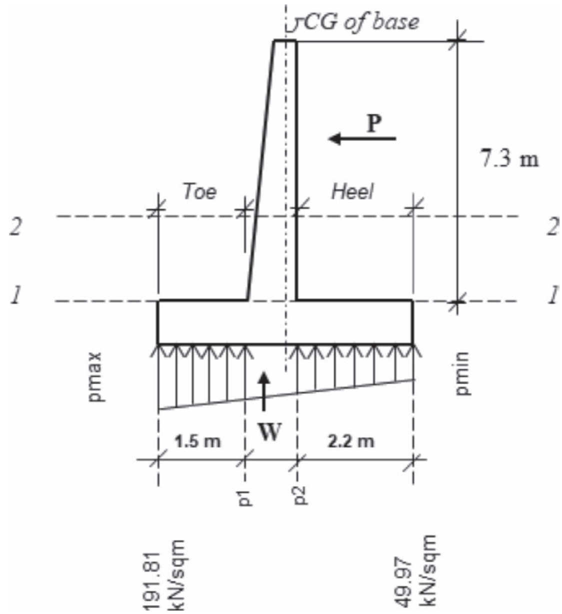

FIGURE 5.60 Base pressure

Maximum pressure, p_{max} = 192 kN/m^2 < 250 kM/m^2; allowable gross bearing pressure.
Minimum pressure, p_{min} = 50 kN/m^2 < 250 kM/m^2; No uplift.
Pressure on the face of the wall – toe side, p_1 = 146 kN/m^2
Pressure on the face of the wall – heel side, p_2 = 119 kN/m^2

e = 0.5 × B – CG = 0.5 × 4.5–1.81 = 0.44 m
p_{max} = (W / B) + [(6 × W × e) / B^2] = (544 / 4.5) + [(6 × 544 × 0.44) / 4.5^2] = 192 kN/m^2 / m
p_{min} = (W / B) – [(6 × W × e) / B^2] = (544 / 4.5) – [(6 × 544 × 0.44) / 4.5^2] = 50 kN/m^2 / m
p_1= pmin + [(pmax – pmin) / B] × (B – Toe)
= 50 + [(192 – 50) / 4.5] × (4.5 – 1.5) = 145 kN/m^2 / m
p_2= pmin + [(pmax – pmin) / B] × (B – Toe – S_{bot})
= 50 + [(192 – 50) / 4.5] × (4.5 – 1.5–0.8) = 119 kN/m^2 / m

Base raft

Net upward pressure will be considered in the strength design of the member section.
Net pressure = gross pressure – self-weight of the base slab

Toe slab

Bending moment, M toe = [0.5 × (192 + 145) – 0.8 × 25] × 1.5^2 / 2 = 167 kNm

Shear, Ftoe = [0.5 × (192 + 145) − 0.8 × 25] × (1.5 − 0.74) = 113 kN
Overall depth of the slab, B_T = 0.8 m cover = 50 mm ϕ dia = 20 mm
Effective depth, d eff = 0.74 m LF = 1.5
[Mtoe]u / b.deff^2 = 1.5 × 167000000 / (1000 x 740^2) = 0.457 (singly reinforced)
Reinforcement required:
p_t = 0.142 % A_{st} = 1051 cm^2 / m width at the bottom.
p_c = 0.2 % A_{sc} = 1480 cm^2 / m width at the top.
[Considering the footing slab acting as a wide beam, minimum reinforcement should be 0.2%.]

Provide bars 20 mm diameter @ 175 mm c/c at the bottom Ast = 1794 > 1051 cm^2/m; safe.
and 20 mm diameter @ 175 mm c/c at the top Ast = 1794 > 1480 cm^2/m; safe.
Shear stress developed, t_v = 1.5 × 113 × 1000 / (1000 × 0.74 × 1000) = 0.23 N/mm^2
Shear stress allowable, t_c = 0.33 N/mm^2 > 0.23; Safe.

Heel

Bending moment, M_{heel} = [(0.5 × (50 + 119) − 0.8 × 25] × (2.2^2 / 2) − (299 × 2.2 / 2) = (−) 172 kNm Tension at the top.
Shear F_{heel} = [0.5 x (50 + 119) − 0.8 x 25] x 2.2−299 = (−) 157 kN.
Overall depth of the slab, B_T = 0.8 m cover = 50 mm ϕ dia = 20 mm
Effective depth, d eff = 0.74 m LF = 1.5
[Mtoe]u / b.deff^2 = 0.472 (singly reinforced.)
Reinforcement required:
p_t = 0.142 % A_{st} = 1051 cm^2 / m width at the bottom.
p_c = 0.2 % A_{sc} = 1480 cm^2 / m width at the top.
Provide Bars 20 mm diameter @ 175 mm c/c at the bottom Ast = 1794 > 1051 cm^2/m; safe.
and 20 mm diameter @ 175 mm c/c at top Ast = 1794 > 1480 cm^2/m; safe.
Shear stress developed, t_v = 1.5 × 157 × 1000 / (1000 × 0.74 × 1000) = 0.318 N/mm^2
Shear stress allowable, t_c = 0.353 N/mm^2 > 0.318; safe.

Stem

At Base – Sec 1–1
Bending moment, Mstem1 = P x (hp − BT) = 149 × (3.18−0.8) = 355 kNm/m (tension at fill 1 side)
Shear Fstem1 = P x (h1 − deff) / h1 = 149 × (7−0.74) / 7 = 133 kN
Overall depth of the slab, B_{T1} = 0.8 m cover = 50 mm ϕ dia = 20 mm
Effective depth, d eff = 0.74 m LF = 1.5
[Mstem1]u / b.deff^2 = 1.5 × 355000000 / (1000 × 740^2) = 0.97 (singly reinforced.)
Reinforcement required:
p_{t1} = 0.291 % A_{st1} = 2153 cm^2 / m width Vertical – interior.
p_{c1} = 0.2 % A_{sc1} = 1480 cm^2 / m width Vertical – exterior.
Provide Bars 25 mm diameter @ 175 mm c/c at internal side Ast = 2804 > 2153 cm^2/m; safe.

and 20 mm diameter @ 175 mm c/c at external side Ast = 1794 > 1480 cm^2/m; safe.

Shear stress developed, t_{v1}= 1.5 × 133 × 1000 / (1000 × 0.74 × 1000) = 0.270 N/mm^2

Shear stress allowable, t_{c1} = 0.386 N/mm^2 > t_{v1}; safe.

At one third of the stem height from the base – Sec 2–2

Height of the stem = 7.2 m ($h_0 + h_1$)

Height at *Sec 2–2* above the base = 2.4 m ($h_0 + h_1$)/3

Height of the wall above *Sec 2–2* = 4.8 m

Bending moment Mstem2 = 355 × (7.2 – 2.4) /7.2 = 237 kNm/m (Tension at fill one side)

Shear Fstem2 = 133 × (7.2 – 2.4) /7.2 = 89 kN/m

Wall thickness at *Sec 2–2,* B_{T2} = 0.4 + [(0.8 – 0.4) × 4.8 / 7.2] = 0.667 m

Cover = 50 mm Effective depth, deff = 0.607 m

Load factor = 1.5

[Mstem2]u / b.deff^2 = 1.5 × 237000000 / (1000 × 607^2) = 0.965 (singly reinforced)

Reinforcement required:

p_{t2} = 0.291 % A_{st2}= 2153 cm^2 / m width Vertical – interior.

p_{c2} = 0.2 % A_{sc2} = 1480 cm^2 / m width Vertical – exterior.

Provide bars 25 mm diameter @ 175 mm c/c at the internal side Ast = 2804 > 2153 cm^2/m; safe.

and 20 mm diameter @ 175 mm c/c at the external side Ast = 1794 > 1480 cm^2/m; safe.

Shear stress developed, t_{v2} = 1.5 × 89000 / (1000 × 607) = 0.22 N/mm^2

Shear stress allowable, t_{c2} = 0.438 N/mm^2 > t_{v2}; safe.

[Strength design of the wall and base slab will be the same as the slab design by ACI 318]

Summary of results:

Stability check –

1. *Factor of safety against overturning: 2.08* *Okay*
2. *Factor of safety against sliding: 2.01* *Okay; shear key not required.*

Strength design –

1. *Base slab – safe in flexure and shear.*
2. *Vertical wall – safe in flexure and shear.*

5.10 DESIGN OF CABLE TRENCH

Cable trenches are used to carry electrical cables below ground in plant buildings and outdoor yard. The outdoor trenches are covered with removable-type precast cover and indoor trenches with removable steel checkered plate floor cover. The trench wall and base are made of reinforced concrete. The bottom of trenches are provided with transverse and longitudinal slope to carry seepage water, if any, towards a designated cable sump pit. The accumulated water is pumped out from sumps to keep the entire network of trenches dry at all seasons.

The cables are laid on steel trays, which are supported from the side wall by cantilever-type steel members at regular intervals. These steel brackets are welded to mild steel insert plates embedded on the wall surface inside the trench. Alternately, these trays can be held and fixed on the wall by using fabricated materials (*unistrut* or equivalent).

Design parameters:

Finish-grade level FGL = 0.0 M Top of trench level TOP = 0.2 M
Groundwater level GWL = (–) 0.5 M Ground surcharge γsurch = 10 kN/m^2
Soil bearing pressure p_{gross} = 80 kN/m^2 Unit weight of soil γsoil = 18 kN/m^3
Submerged wt of soil γsub = 8 kN/m^3 Unit weight of concrete, γconc =25 kN/m^3
Coefficient of earth pr. at rest, K_0 = 0.5

Grade of concrete M25 fck = 25 N/mm^2
Reinforcement bars fy = 415 N/mm^2
Concrete cover = 40 mm Maximum bar diameter, ϕ max = 12 mm
Weight of a cable tray, w_{tray} = 1.2 kN/m Center distance of the tray, e_{tray} = 0.40 m
Precast cover: Width = 0.6 m (600 mm); Thickness = 0.04 m (40 mm)

Types of trenches

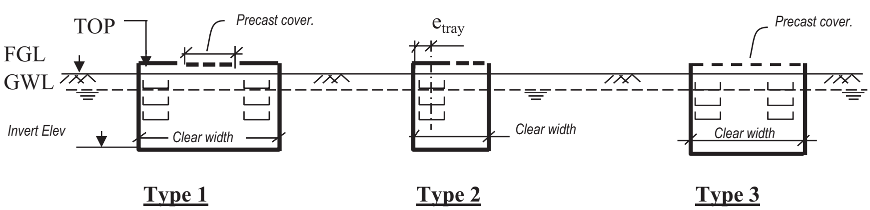

FIGURE 5.61 Types of cable trenches

Dimensions and sizing

TABLE 5.63
Dimensions of cable trenches

Trench mkd	Type	Inv EL (Elev) M	Clear width M	No. of trays Left	No. of trays Right	Wall / slab thickness Top m	Side m	Base m
CT1	1	−1.4	1.8	4	4	0.125	0.225	0.225
CT2	2	−1.4	1.2	4	4	0.125	0.225	0.225
CT3	3	−1.25	1.2	3	3	0.125	0.225	0.225

Reinforcement bars

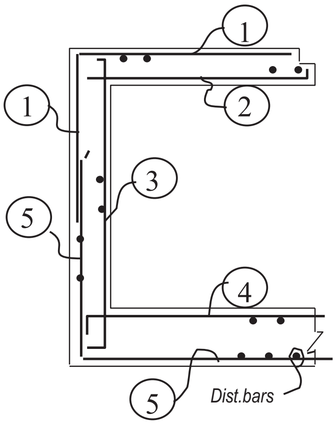

FIGURE 5.62 Reinforcement bar details

TABLE 5.64
Reinforcement bar descriptions

	Bar mark and details											
	1		2		3		4		5		Dist. bars	
Trench	dia	spcg	dia	spcg	dia	spcg	dia	spcg	dia	spcg	dia	spcg
mkd	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
CT1	8	200	8	200	10	200	10	200	10	200	8	200
CT2	8	200	8	200	10	200	10	200	10	200	8	200
CT3	10	200	10	200	10	200	10	200	10	200	8	200

Note: Distribution bars or binders are same at base, wall and top slab.

Loading diagram

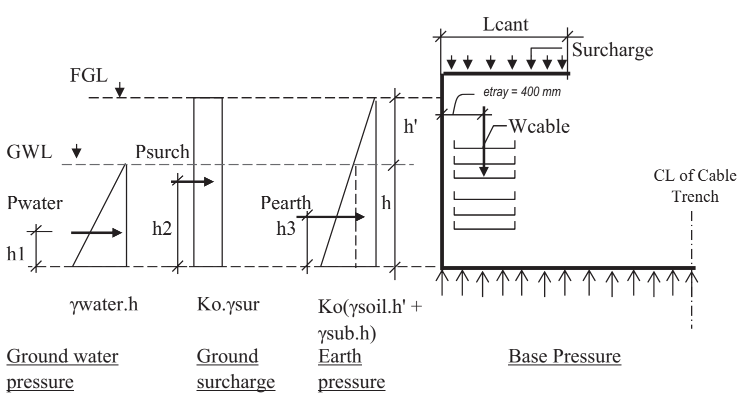

FIGURE 5.63 Loading on cable trenches

TABLE 5.65
Design calculation

Trench mark — **Calculation**

CT1

Type 1 *Vertical load:*

Self-weight:

Top slab =	$2 \times 0.6 \times 0.125 \times 25 =$	3.75	kN/m
Precast cover =	$0.6 \times 0.04 \times 25 =$	0.6	kN/m
Side walls =	$2 \times (0.2+1.4) \times 0.225 \times 25 =$	18	kN/m
Base slab =	$(1.8+ 2 \times 0.225) \times 0.225 \times 25 =$	12.7	kN/m
		35.0	kN/m

Wselfwt 35.00 kN/m

Total surcharge = $10 \times (1.8 + 2 \times 0.225) = 22.50$ kN/m

Wcable (left) = $4 \times 1.2 = 4.8$ kN/m

Wcable (right) = $4 \times 1.2 = 4.8$ kN/m

Lateral pressure:

Psurch $=0.5 \times 10 \times 1.4 = 7$ kN/m

Pearth = 6.80 KN/m

h =	GWL − Inv EL	0.9	m
h′ =	FGL − GWL =	0.5	m

Pearth = $(0.5 \times 0.5 \times 18 \text{ x } 0.5^2) + (0.5 \times 0.5 \times 8 \text{ x } 0.9^2) + (0.5 \times 18 \times 0.5 \times 0.9)$

= 6.80 kN/m

(*Continued*)

TABLE 5.65 (*Continued*)
Design calculation

Trench mark | **Calculation**

Pwater = $10 \times 0.9^2 \times 0.5 = 4.05$ kN/m

Check for uplift: per m long

Gr. water pr. = (GWL – Inv EL + base thickness) × 10 = 11.25 kN/m^2

Upward thrust = Groundwater pressure × Base area = $11.25 \times (1.8 + 2 \times 0.225)$

= 25.31 kN/m run

F.O.S = Wselfwt (without cover) / Upward thrust = (35 – 0.6) / 25.31

= 1.36 < 1.2; Safe.

Base pressure: per m long

Total downward load = Wselfwt + Surcharge + Wcable

= 35 + 22.5 + 4.8 + 4.8 = 67.10 kN

Base area = $1.8 + 2 \times 0.225$ = 2.25 Sqm

Base pressure = 67.1 / 2.25 = 29.82 kN/m^2

< SBP; Safe

Strength design check by the limit state method – IS 456: 2000 & SP 16

Design of the top slab:

Load intensity: 10.00 kN/m^2 (surcharge) and 3.125 kN/m^2 (weight of slab)

UDL = 10 + 3.125 = **13.13** kN/m (0.125 × 25)

Lcant = (1.8 – 0.04) / 2 = 0.6 m

Mmax = (13.125 × 0.6^2) / 2 = 2.36 kNm/m Load factor = **1.5**

Effective depth, d = 0.125 × 1000 − (40 + 12 × 0.5) / 1000 = 0.079 m

Mu = 1.5 × 2.36 = 3.54 kNm

M_ULimit/bd^2 = 3.45 (Table – D SP 16)

M_U Limit = 22 kNm **Singly reinforced**

Mu / bd^2 = 3540000 / (1000 × 79^2) = 0.57

Bar – 1

Reinforcement percentage: (SP 16 – Table 30)

% pt 0.12 Minimum reinforcement

Ast reqd = (0.12 × 1000 × 79) / 100 = 95 mm^2 / m.

Ast provided: 8 diameter @ 200 mm centers = 251 mm^2/m Safe.

Bar 2

% pt 0.12 Minimum reinforcement

Ast reqd = (0.12 × 1000 × 79) / 100 = 95 mm^2 / m.

Ast provided: 8 dia@ 200 mm centers = 251 mm^2/m Safe.

Design of the side wall:

External face

Load: Pwater + Psurch + Pearth + Pcable + topslab

Projection above the base = TOP – Inv EL = 0.2 – (−1.4) = 1.6 m

Wall moment at the external face, Msidewallext:

h1 = (GWL – Inv EL) / 3 = [−0.5 – (−1.4)] / 3 = 0.3 m

(*Continued*)

TABLE 5.65 (*Continued*)
Design calculation

Trench mark	Calculation					
	h2 =	(FGL – Inv EL) / 2 = [0 – (–1.4)] / 2 = 0.7 m				
	h3 =	(FGL – Inv EL) / 3 = [0 – (–1.4)] / 3 = 0.47 m				
	L cant =	0.6 m				
	Msidewallext =	$4.05 \times 0.3 + 7 \times 0.7 + 6.8 \times 0.47 + 13.125 \times 0.6^2 / 2 + 4.8 \times 0.4$				
	= 14 kNm					
	d =	225 – 40 – 6 =	179 mm	LF =	**1.5**	
	Mu =	1.5 x 14 =	21 kNm			
	$M_U Limit/bd^2$ =	3.45			(Table D SP16)	
	M_U Limit =	111 kNm.	**Singly reinforced**			
	Mu / bd^2 =	$21000000 / (1000 \times 179^2) = 0.66$				
	Bar 5					
	Reinforcement percentage:				(SP 16 – Table 3)	
	% pt	0.20				
	Ast reqd =	$(0.2 \times 1000 \times 179) / 100 =$ **358** mm^2 / m.				
	Ast provided:					
	Bar 5	10 diameter @	200 mm centers = **393** mm^2/m			Safe
	Internal face:					
		Load:	Pwater (*)			
		Lcant	0.60 m	d	179 mm	

Moment inside face of wall = Pwater × h1 − UDL. $Lcant^2$ /2

Msidewallint = 4.05 × 0.3 = 1.22 kNm

Mu = 1.5 × 1.22 = 1.83 kNm LF = 1.5

Mu / bd^2 = 1830000 / (1000 x 179^2) = 0.06

pt = 0.12 %

Ast reqd = 0.12 × 1000 × 179/100 = **215** mm^2/m

Bar 3 Ast prov 10 diameter @ 200 mm centers = **393** mm^2/m Safe.

(*) Groundwater inside for worst case

Design of the base slab:

Load: Net base pressure upwards = 29.82 − 0.225 × 25 = 24.20 kN/m^2

Effective depth, d = 225 − 40 − 0.5 × 12 = 179 mm

Effective span = 1.8 + 0.179 = 1.98 m

Mspan = (24.2 × 1.98^2) / 10 9.49 kNm

Msupp = (24.2 × 1.98^2) / 12 7.91 kNm

M_ULimit = 111 kNm. Singly reinforced M_ULimit/bd^2 = 3.45

Mu = 1.5 x 9.49 = 14.24 kNm LF = 1.5

Mu / bd^2 = 0.444

Bar 4

% pt 0.12

Astrequired = 0.12 × 1000 × 179 / 100 = 215 mm^2 / m.

(*Continued*)

TABLE 5.65 (*Continued*)
Design calculation

Trench mark	Calculation					
	Ast provided		10 dia@	200 mm centers =	393 mm^2/m	Safe.
	Bar 5					
	% pc	0.12	Asc =	215 mm^2 / m.		
	Ast provided		10 diameter @ 200 mm centers =	393 mm^2/m		Safe.

Strength design check in the LRFD method – ACI 318–19

Load combination:	1.2 D + 1.6 L				(ACI 318 – Table 5.3.1)
	where	D =	dead load		
		L =	surcharge, earth and hydrostatic pressure		

$C = T$

or $0.85fc'. a. bw = As.fy$

or $a = As.fy /(0.85\ fc'\ bw)$

$\phi\ Mn = \phi\ As\ fy\ (d - a/2) > Mu$

Top slab:

Load intensity:	10.00 kN/m^2	(surcharge)	and	3.125 kN/m^2	(weight of slab)

UDL = 1.2 × 3.125 + 1.6 × 10 = 19.75 kN/m

Load factor for:	
Dead load, D	1.2
Others, L	1.6

Lcant = 0.6 m

Mu = $(19.75 \times 0.6^2) / 2 = 3.56$ kNm/m   Factored

Effective depth, d =	$0.125 \times 1000 - (40 + 12 \times 0.5) = 79$ mm					
Strength reduction factor,		ϕ	0.9		ACI 318–19 (Table 21.2.1)	
		Mu	**3.56**	kNm		
Required nominal strength, M_n =		Mu/ϕ	3.96	kNm		
fc′ = 20 N/mm²	(= 0.8fck)	bw =	1000	mm		
fy = 415 Mpa		D =	125	mm		
		d =	79	mm		
Bar 1 and 2						
Let us try with	8 mm diameter bars@	200	c/c.	Ast =	251 mm²	
					> Min reinforcement.	
Ast min =	142 mm²/m width	$(0.18 \times 1000 \times 79/100)$				
a =	$251 \times 415 / (0.85 \times 20 \times 1000) =$		6 mm			
ϕ Mn =	ϕ As fy (d − a/2) =	$0.9 \times 251 \times 415 \times (79 - 6/2)/10^6 = 7$ kNm				
					> Mu; Safe.	
Provide reinforcement at top and bottom faces:		8 mm diameter bars@		200 c/c.		*fos*= 2

(*Continued*)

TABLE 5.65 (*Continued*)
Design calculation

Trench mark	Calculation							
			Ast provided =		251	mm^2	**Safe.**	
	Design of side wall:							LF:
	External face							1.2 D
	Load:	Pwater + Psurch + Pearth + Pcable + topslab (factored)						1.6 L
		Factored load = 1.2D+1.6L						
		Pwater =	4.05	×	1.6	=	6.48 kN	
		Psurch =	7	×	1.6	=	11.2 kN	
		Pearth =	6.80	×	1.6	=	10.88 kN	
		Wcable =	4.8	×	1.6	=	7.68 kN	
		UDL (top slab)=			19.75 kN/m^2	(factored)		
		h1 =	0.3 m	h2 =	0.7 m	h3 =		0.47 m

Msidewallext = Pwater × h1 + Psurch × h2 + Pearth × h3 + 0.5 × UDL × Lcant ^{2} + Wcable × etray

Msidewallext

= 6.48 × 0.3 + 11.2 × 0.7 + 10.88 × 0.47 + 19.75 × 0.6^2 / 2 + 7.68 × 0.4 = 22 kNm ACI 318–19 (Table 21.2.1)

Strength reduction factor, ϕ 0.9

Mu **22.00** kNm

Required nominal strength, $M_n = Mu/\phi = 24.44$

fc′ = 20 Mpa

fy = 415 Mpa

bw =	1000	mm
D =	225	mm
d =	179	mm

Bar 5 10 diameter @ 200 mm centers = 393 mm^2/m

Ast min = $0.18 \times 1000 \times 179/100 = 322$ mm2/m width

a = $393 \times 415 / (0.85 \times 20 \times 1000) = \times 10$ *mm*

$\phi Mn = \phi As\, fy\, (d - a/2) = 0.9 \times 393 \times 415 \times (179 - 10/2)/10^6 = 26$ *kNm > Mu; Safe.*

Internal face:

Load: Pwater inside

Moment inside the face of the wall = Pwater × h1

Msidewallint = 6.48 × 0.3 = 1.94 kNm

Strength reduction factor,	ϕ	0.9		ACI 318–19 (Table 21.2.1)
	Mu	**1.94**	kNm	
Required nominal strength, $M_n = Mu/\phi$ 2.16 kNm				
fc′ = 20 N/mm^2	bw =	1000	Mm	

(*Continued*)

TABLE 5.65 (*Continued*)
Design calculation

Trench mark	Calculation					
	fy =	415	Mpa		D = 225 Mm	
					d = 179 Mm	
	Bar 3					
	let us try with		10 mm diameter bars@ 200 c/c.		Ast = 393 mm²	
						> Minimum reinforcement
	Ast min =	322	mm²/m width		$(0.18 \times 1000 \times 179/100)$	
	a =	$393 \times 415 / (0.85 \times 20 \times 1000) = 10$ mm				
	ϕ *Mn =*	$\phi\ As\ fy\ (d - a/2) =$			$0.9 \times 393 \times 415 \times (179 - 10/2)/10^6 = 26$ kNm	
						> Mu; safe
	Provide reinforcement at top and bottom faces:			10 mm diameter bars@ 200 c/c.		
				Ast provided =	393 mm²	**Safe**
	Design of the base slab:					
	Load:	Net base pressure upwards =			24.20 kN/m²	

	Effective depth, d =		179	mm		
	Effective span =		1.98	m		
	Mspan =		9.49	kNm		
	Msupp =		7.91	kNm		
	Mu = 1.6×9.49 =		15	kNm		LF = 1.6
Bar 4						
Strength reduction factor,		ϕ	0.9		ACI 318–19 (Table 21.2.1)	
		Mu	**15.00**	kNm		
Required nominal strength, M_n =		Mu/ϕ	16.67	kNm		
fc′ =	20 N/mm^2	bw =	1000	mm		
fy =	415 Mpa	D =	225	mm		
		d =	179	mm		
Bar 4						

(*Continued*)

TABLE 5.65 (*Continued*)
Design calculation

Trench mark	**Calculation**
	let us try with 10 mm diameter bars@ 200 c/c. Ast = 393 mm^2
	> Min reinforcement.
	Ast min = 322 mm^2/m width (0.18 × 1000 × 179/100)
	$a = 393 \times 415 / (0.85 \times 20 \times 1000) =$ 10 mm
	$\phi Mn = \phi As\, fy\, (d - a/2) =$ $0.9 \times 393 \times 415 \times (179 - 10/2) / 10^6 = 26$ kNm
	> Mu; Safe. fos = 1.7
	Provide reinforcement at top and bottom faces: 10 mm diameter bars@ 200 c/c.
	Ast provided= 393 mm^2 **Safe.**
CT2	
Type 2	*Vertical load:*
	Self-weight:
	Top slab = 0.6 × 0.125 × 25 = 1.88 kN/m
	Precast cover = 0.6 × 0.04 × 25 = 0.60 kN/m
	Side walls = 2 × (0.2+1.4) × 0.225 × 25 = 18.00 kN/m
	Base slab = (1.2 + 2 × 0.225) × 0.225 × 25 = 9.28 kN/m
	29.76 kN/m
	Wselfwt = 29.76 kN/m

Total surcharge = $10 \times (1.2 + 2 \times 0.225) = 16.50$ kN/m

Wcable – on the left =	$4 \times 1.2 =$	4.8	kN/m
Wcable – on the right =	$4 \times 1.2 =$	4.8	kN/m

Lateral pressure:

Psurch = $0.5 \times 10 \times 1.4 = 7$ kN/m

Pearth =	6.80 kN/m	h =	GWL – Inv EL	0.9	m
		h′ =	FGL – GWL =	0.5	m

Pearth = $(0.5 \times 0.5 \times 18 \times 0.5^2) + (0.5 \times 0.5 \times 8 \text{ x } 0.9^2) + (0.5 \times 18 \times 0.5 \times 0.9)$

= 6.80 kN/m

Pwater = $10 \times 0.9^2 \times 0.5 = 4.05$ kN/m

Check for uplift: per meter length

Gr. water pr. = (GWL – Inv EL + base thickness) $\times 10 = 11.25$ kN/m^2

Upward thrust = Groundwater pressure × Base area= $11.25 \times (1.2 + 2 \times 0.225)$

= 18.56 kN/m run

F.O.S = Wselfwt (without cover)/Upward thrust = $(29.76 - 0.6) / 18.56$

= $1.57 < 1.2$; Safe.

Base pressure: per meter length

(*Continued*)

TABLE 5.65 (*Continued*)
Design calculation

Trench mark	Calculation

Total downward load = Wselfwt + Surcharge + Wcable

= 29.76 + 16.5 + 4.8 + 4.8 = 55.86 kN

Base area = 1.2 + 2 × 0.225 = 1.65 Sqm

Base pressure = 55.86 / 1.65 = 33.85 kN/m^2

< SBP; Safe

Strength design check by the limit state method – IS 456: 2000 & SP 16

Design of top slab:

Load intensity: 10.00 kN/m^2 (surcharge) and 3.125 kN/m^2 (weight of slab)

UDL = 10 + 3.125 = **13.13** kN/m

Lcant = 1.2 / 2 = 0.6 m

Mmax = $(13.125 \times 0.6^2) / 2 = 2.36$ kNm/m Load factor = **1.5**

Effective depth, d = $0.125 \times 1000 - (40 + 12 \times 0.5) = 79$ mm

Mu = $1.5 \times 2.36 =$ 3.54 kNm

M_U Limit/bd^2 = 3.45 (Table D SP 16)

M_U Limit = 22 kNm. **Singly reinforced**

Mu / bd^2 = $3540000 / (1000 \text{ x } 79^2) = 0.57$

Bar – 1

Reinforcement percentage: (SP 16 – Table 30)

% pt 0.12 Minimum reinforcement

Ast reqd = $(0.12 \times 1000 \times 79) / 100 = 95$ mm^2 / m.

Ast provided: 8 dia@ 200 mm centers = 251 mm^2/m Safe.

Bar 2

% pt 0.12 Minimum reinforcement
Ast reqd = (0.12 × 1000 × 79) / 100 = 95 mm^2 / m.
Ast provided: 8 dia@ 200 mm centers = 251 mm^2/m Safe.

Design of the side wall:
External face
Load: Pwater + Psurch + Pearth + Pcable + topslab
Projection above the base = TOP – InvEL= 0.2 – (–1.4) = 1.6 m

Wall moment at the external face, Msidewallext:
h1 = (GWL – Inv EL) / 3 = [–0.5 – (–1.4)] / 3 = 0.3 m
h2 = (FGL – Inv EL) / 2 = [0 – (–1.4)] / 2 = 0.7 m
h3 = (FGL – Inv EL) / 3 = [0 – (–1.4)] / 3 = 0.47 m
L cant = 0.6 M

Msidewallext = $4.05 \times 0.3 + 7 \times 0.7 + 6.8 \times 0.47 + 13.125 \times 0.6^2 / 2 + 4.8 \times 0.4$
= 14 kNm
d = 225 – 40 – 6 = 179 mm LF = **1.5**
Mu = 1.5 × 14 = 21 kNm
M_ULimit/bd^2 = 3.45 (Table D SP16)
M_ULimit = 111 kNm. **Singly reinforced**
Mu / bd2 = $21000000 / (1000 \times 179^2) = 0.66$

Reinforcement percentage: (SP 16 – Table 3)

(*Continued*)

TABLE 5.65 (*Continued*)
Design calculation

Trench mark	Calculation						
	% pt	0.20					
	Ast reqd =		$(0.2 \times 1000 \times 179) / 100 = 358\ mm^2 / m$.				
	Ast provided:						
	Bar 5	10	diameter @	200 mm centers = $393\ mm^2 / m$.			Safe.
	Internal face:						
		Load:	Pwater (*)				
		Lcant	0.60	m	d	179 mm	
	Moment inside face of wall = Pwater × h1						
	Msidewallint =		$4.05 \times 0.3 =$			1.22 kNm	
	Mu =	$1.5 \times 1.22 = 1.83$ kNm			LF =	**1.5**	
	$Mu / bd^2 =$		$1830000 / (1000 \times 179^2) = 0.06$				
	pt =	0.12	%				
	Ast reqd =		$0.12 \times 1000 \times 179/100 = 215\ mm^2/m$				
	Bar 3	Ast provided:	10 dia@ 200 mm centers = $393\ mm^2/m$				Safe.
	(*) Groundwater inside for worst case						
	Design of base slab:						

Load:	Net base pressure upwards = $33.85 - 0.225 \times 25 = 28.23$ kN/m^2				
	Effective depth, d =		$225 - 40 - 0.5 \times 12 =$	179 mm	
	Effective span =		$1.2 + 0.179 =$	1.38 m	
	Mspan =	$(28.23 \times 1.38^2) / 10$		5.38 kNm	
	Msupp =	$(28.23 \times 1.38^2) / 12$		4.48 kNm	
M_ULimit =	111 kNm.	**Singly reinforced**		M_ULimit/bd^2 =	3.45
Mu =	1.5 x 5.38 =	8.07 kNm		LF = 1.5	
Mu / bd^2	=0.252				
Bar 4					
% pt	0.12				
	Ast required =	$0.12 \times 1000 \times 179 / 100 = 215$ mm^2 / m.			
Ast provided	10	dia@ 200 mm centers = 393 mm^2/m			Safe.
Bar 5					
% pc	0.12	Asc = 215 mm^2/m.			
Ast provided	10	dia@	200 mm centers =	393 mm^2/m	Safe.

Strength design check in the LRFD method – ACI 318–19

Load combination: 1.2 D + 1.6 L (ACI 318 – Table 5.3.1)

where D = dead load
L = surcharge, earth and hydrostatic pressure

Refer to for stress block diagram of slab (Figure 5.35)

(*Continued*)

TABLE 5.65 (*Continued*)
Design calculation

Trench mark | **Calculation**

$C = T$

Or $0.85fc'. a. bw = As.fy$

Or $a = As.fy /(0.85 fc' bw)$

$\phi Mn =$ $\phi As fy (d - a/2) > Mu$

Top slab:

	Load intensity:	10.00	kN/m^2	(surcharge)	and	3.125	kN/m^2	(weight of the slab)
					Load factor for			
UDL =	$1.2 \times 3.125 + 1.6 \times 10 =$			19.75 kN/m	Dead load, D			1.2
					Others, L			1.6
Lcant =	0.6	m						
Mu=	$(19.75 \times 0.6^2) / 2 =$		3.56	kNm/m		Factored		
Effective depth, d =	$0.125 \times 1000 - (40 + 12 \times 0.5) = 79$ mm							
Strength reduction factor,			ϕ	0.9		ACI 318–19 (Table 21.2.1)		
			Mu	**3.56**	kNm			
Required nominal strength, M_n =			Mu/ϕ	3.96	kNm			

fc′ = 20 N/mm² bw = 1000 mm
fy = 415 Mpa D = 125 mm
d = 79 mm

Bars 1 and 2

Let us try with 8 mm diameter bars@ 200 c/c. Ast = 251 mm²
>Minreinforcement.

Ast min = 142 mm²/m width (0.18 × 1000 × 79/100)

a = 251 × 415 / (0.85 × 20 × 1000) = 6 mm
ϕ Mn = ϕ As fy (d − a/2) = 0.9 × 251 × 415 × (79 − 6/ 2) / 106 = 7 kNm

> Mu; Safe.

Provide reinforcement *fos* = 2
at top and bottom faces: 8 mm diameter bars @ 200 c/c.

Ast provided = 251 mm² **Safe.**

Design of the side wall:
External face LF = 1.2 DL + 1.6 LL
Load: Pwater + Psurch + Pearth + Pcable + topslab

Pwater = 4.05 × 1.6 = 6.48 kN

(*Continued*)

TABLE 5.65 (*Continued*)
Design calculation

Trench mark	Calculation

Psurch =	7	×	1.6	=	11.2 kN
Pearth =	6.80	×	1.6	=	10.88 kN
Wcable =	4.8	×	1.6	=	7.68 kN
UDL (top slab)=			19.75 kN/m²	(factored)	

h1 = 0.3 m h2 = 0.7 m h3 = 0.47 m

Msidewallext = Pwater × h1 + Psurch × h2 + Pearth × h3 + 0.5 × UDL × Lcant ^{2} + Wcable × etray

Msidewallext = $6.48 \times 0.3 + 11.2 \times 0.7 + 10.88 \times 0.47 + 19.75 \times 0.6^2 / 2 + 7.68 \times 0.4$
= 22 kNm

				ACI318–19
Strength reduction factor,	ϕ	0.9		(Table21.2.1)
	Mu	**22.00**	kNm	
Required nominal strength, M_n =	Mu/ϕ	24.44	kNm	
fc′ = 20 N/mm²	bw =	1000	mm	
fy = 415 Mpa	D =	225	mm	
	d =	179	mm	

Bar 5	10 diameter @	200 mm centers =	393	mm²/m
		Total =	393	mm²/m

Ast min = 0.18 × 1000 × 179/100 = 322 mm^2/m width

a = *393 × 415 / (0.85 × 20 × 1000) =* *10 mm*

ϕ *Mn =* ϕ *As fy (d – a/2) = 0.9 × 393 × 415 × (179 – 10 / 2) / 10^6 =26 kNm*

> Mu; Safe.

Internal face:

Load: Pwater inside

Moment inside face of wall = Pwater × h1

Msidewallint = 6.48 × 0.3 = 1.94 kNm

ACI 318–19
(Table 21.2.1)

Strength reduction factor,	ϕ	0.9	
	Mu	**1.94**	kNm
Required nominal strength, M_n =	Mu/ϕ	2.16	kNm

fc′ =	20	N/mm^2	bw =	1000	mm
fy =	415	Mpa	D =	225	mm
			d =	179	mm

Bar 3

let us try with 10 mm diameter bars@ 200 c/c. Ast = 393 mm^2

(*Continued*)

TABLE 5.65 (*Continued*)
Design calculation

Trench mark	Calculation					
					> Min reinforcement.	
	Ast min =	322	mm²/m width	(0.18 × 1000 × 179/100)		
	a =	*393 × 415 / (0.85 × 20 × 1000) = 10 mm*				
	ϕ Mn =	*ϕ As fy (d – a/2) = 0.9 × 393 × 415 × (179 – 10 / 2) / 106 =26 kNm*				
					> Mu; Safe.	*13.2 fos*
	Provide reinforcement					
	At top and bottom faces:		10 mm diameter bars @	200	c/c.	
		Ast provided	393	mm²	**Safe.**	
	Design of the base slab:					
	Load:	Net base pressure upwards =	28.23	kN/m²		
		Effective depth, d =	179	mm		
		Effective span =	1.38	m		
		Mspan =	5.38	kNm		
		Msupp =	4.48	kNm		
		Mu =	1.6 × 5.38 = 9 kNm			LF = 1.6
	Bar 4					

Strength reduction factor,		ϕ	0.9	ACI 318–19 (Table 21.2.1)
		Mu	**9.00** kNm	
Required nominal strength, M_n =		Mu/ϕ	10.00 kNm	
fc′ = 20 N/mm²		bw =	1000 mm	
fy = 415 Mpa		D =	225 mm	
		d =	179 mm	
Bar 4				
Let us try with	10 mm diameter bars@ 200 c/c.	Ast =		393 mm² > Min reinforcement.
Ast min =	322 mm²/m width	(0.18 × 1000 × 179/100)		
a =	*393 × 415 / (0.85 × 20 × 1000) =*		*10 mm*	
ϕ *Mn =*	ϕ *As fy (d – a/2) =*	*0.9 × 393 × 415 × (179 – 10 / 2) / 106 =26 kNm*		
				> Mu; Safe.
Provide reinforcement at top and bottom faces:		10 mm diameter bars @ 200 c/c.		
	Ast provided = 393 mm²			**Safe.**

(*Continued*)

TABLE 5.65 (*Continued*)
Design calculation

Trench mark		Calculation
CT3	**3**	*Vertical load:*

Self-weight:

Top cover (precast) =	$1.65 \times 0.125 \times 25 =$	5.16	kN/m
Side walls =	$2 \times (0.2+1.25) \times 0.225 \times 25 =$	16.31	kN/m
Base slab =	$1.65 \times 0.225 \times 25 =$	9.28	kN/m
		30.75	kN/m

Wselfwt 30.75 kN/m

Total surcharge= $10 \times (1.2 + 2 \times 0.225) =$ 16.50 kN/m

Wcable (left) = $3 \times 1.2 =$ 3.6 kN/m

Wcable (right) = $3 \times 1.2 =$ 3.6 kN/m

Lateral pressure:

Psurch = $0.5 \times 10 \times 1.25 =$ 6.25 kN/m

Pearth = 5.63 kN/m

h = GWL – Inv EL 0.75 m

h′ = FGL – GWL = 0.5 m

Pearth =

$= (0.5 \times 0.5 \times 18 \times 0.5^2) + (0.5 \times 0.5 \times 8 \times 0.75^2) + (0.5 \times 18 \times 0.5 \times 0.75)$

$= 5.63$ kN/m

Pwater = $10 \times 0.75^2 \times 0.5 = 2.81$ kN/m

Check for Uplift: per meter length

Gr. water pr. = (GWL – Inv EL + base thickness) x 10 = 9.75 kN/m^2

Upward thrust = Ground pressure × Base area = 9.75 × (1.2 + 2 × 0.225)

= 16 kN/m run

F.O.S = Wselfwt (without cover)/Upward thrust = (30.75 – 5.16) / 16

= 1.60 < 1.2; Safe.

Base pressure: per meter length

Total downward load = Wselfwt + Surcharge + Wcable

= 30.75 + 16.5 + 3.6 + 3.6 = 54.45 kN

Base area = 1.2 + 2 × 0.225 = 1.65 Sqm

Base pressure = 54.45 / 1.65 = 33.00 kN/m^2

< SBP; Safe

Strength design check by the limit state method – IS 456: 2000 & SP 16

Design of Top slab:

(Simply supported slab acting as a wide beam and spanning over the side walls.)

Load intensity: 10.00 kN/m^2 (surcharge) and 3.13 kN/m^2 (weight of slab)

UDL = 10 + 3.125 = **13.13** kN/m

Span = 1.2 + 0.225 = 1.425 m

Mmax = (13.13 × 1.425^2) / 8 = 3.33 kNm/m Load factor = **1.5**

Effective depth, d = 0.125 × 1000 – (40 + 12 × 0.5) = 79 mm

Mu = 1.5 × 3.33 = 5.00 kNm

M_ULimit/bd^2 = 3.45 (Table D SP 16)

M_ULimit = 22 kNm. **Singly reinforced**

(*Continued*)

TABLE 5.65 (*Continued*)
Design calculation

Trench mark	Calculation						
	$Mu/bd^2 =$	5000000 / (1000 × 79^2) =		0.80			
	Bar – 2 – at the bottom layer						
	Reinforcement percentage:			(SP 16 – Table 30)			
	% pt	0.25					
	Ast reqd =	(0.25 × 1000 × 79) / 100 =		198	mm^2 / m.		
	Ast provided:	10 dia@	200 mm centers =		393	mm^2/m	Safe.
	Bar 1 – at top layer						
	% pt	0.20					
	Ast reqd =	(0.2 × 1000 × 79) / 100 =		158	mm^2 / m.		
	Ast provided:	10 dia@	200 mm centers =		393	mm^2/m	Safe.
	Design of the side wall:						
	External face						
	Load:	Pwater + Psurch + Pearth + Pcable + topslab					
	Projection above base =		TOP InvEL = 0.2 – (−1.25) =			1.45	m
	Wall moment at the external face, Msidewallext:						
	h1 =	(GWL – Inv EL) / 3 =	[−0.5 – (−1.25)] / 3 =			0.25	m
	h2 =	(FGL – Inv EL) /2 =	[0 – (−1.25)] / 2 =			0.63	m
	h3 =	(FGL – Inv EL) / 3 =	[0 – (−1.25)] / 3 =			0.42	m
	Msidewallext =	2.81 × 0.25 + 6.25 × 0.63 + 5.63 × 0.42 + 3.6 × 0.4					
	=	8	kNm				

d = 225 – 40 – 6 = 179 mm LF = **1.5**

Mu = $1.5 \times 8 =$ 12 kNm

M_U limit/bd^2 = 3.45 (Table D SP16)

M_U limit = 111 kNm. **Singly reinforced**

Mu / bd^2 = 12000000 / (1000 × 179^2) = 0.37

Bar – 5

Reinforcement percentage: (SP 16 – Table 3)

% pt 0.12

Ast reqd = (0.12 × 1000 × 179) / 100 = 215 mm^2 / m.

Ast provided:

Bar 5 10 diameter @ 200 mm centers = 393 mm^2/m Safe.

Internal face:

Load: Pwater (inside water + no backfill outside during construction stage)

h1 = 0.25 m d = 179 mm

Moment inside face of the wall = Pwater × h1

Msidewallint = 2.81 × 0.25 = 0.7 kNm

Mu = 1.5 × 0.7 = 1.05 kNm LF = **1.5**

Mu / bd^2 = 1050000 / (1000 × 179^2) = 0.03

pt = 0.12 %

Ast reqd = 0.12 × 1000 × 179/100 = 215 mm^2/m

(*Continued*)

TABLE 5.65 (*Continued*)
Design calculation

Trench mark	Calculation							
	Bar 3	Ast prov	10	diameter @	200 mm centers =	393 mm²/m		Safe.
	Design of the base slab:							
		Load:	Net base pressure upwards = 33 – 0.225 × 25 = 27.38 kN/m²					
			Effective depth, d =		225 – 40 – 0.5 × 12 =		179 mm	
			Effective span =		1.2 + 0.179 =		1.38 m	
			Mspan =		$(27.38 \times 1.38^2) / 10$		5.21 kNm	
			Msupp =		$(27.38 \times 1.38^2) / 12$		4.35 kNm	
		M_U Limit =	111	kNm.	**Singly reinforced**	M_ULimit/bd² =		3.45
		Mu =	1.5 × 5.21 = 7.82 kNm			LF =	1.5	
		Mu / bd²		=0.244				
		Bar 4						
		% pt		0.12				
			Ast required =	0.12 × 1000 × 179 / 100 = 215 mm² / m.				
		Ast provided		10 dia@	200 mm centers =	393	mm²/m	Safe.
		Bar 5						
		% pc	0.12	Asc =	215 mm²/m.			
		Ast provided		10 dia@	200 mm centers =	393	mm²/m	Safe.
	Strength design check using the LRFD method – ACI 318–19							
	Load combination:			1.2 D + 1.6 L		(ACI 318 – Table 5.3.1)		

where D = dead load
L = surcharge, earth and hydrostatic pressure

Refer stress block diagram of slab (Figure 5.35)

$C = T$

or $0.85fc'.\ a.\ bw = As.fy$

or $a = As.fy\ /(0.85\ fc'\ bw)$

$\phi\ Mn =$ $\phi\ As\ fy\ (d - a/2)$ $> Mu$

Top slab:

Load intensity:	10.00	kN/m^2	(surcharge)	and	3.125	kN/m^2	(weight of slab)
					Load factor for:		
UDL =	$1.2 \times 3.125 + 1.6 \times 10 =$		19.75 kN/m		Dead load, D		1.2
					Others, L		1.6
Span =	1.425	m					

Mu = $(19.75 \times 1.425^2) / 8 =$ 5.01 kNm/m Factored

(*Continued*)

TABLE 5.65 **(*Continued*)**
Design calculation

Trench mark	Calculation				
	Effective depth, d =	$0.125 \times 1000 - (40 + 12 \times 0.5) = 79$ mm			
	Strength reduction factor,		ϕ	0.9	ACI 318–19 (Table 21.2.1)
			Mu	**5.01** kNm	
	Required nominal strength, M_n =		Mu/ϕ	5.57 kNm	
	fc′ = 20 N/mm^2		bw =	1000 mm	
	fy = 415 Mpa		D =	125 mm	
			d =	79 mm	
	Bar 1 at the top and Bar 2 at the bottom: simply supported slab spanning over side walls				
	Let us try with	10 mm diameter bars@	200 c/c.	Ast =	393 mm^2 >Minreinforcement.
	Ast min =	142 mm^2/m width	$(0.18 \times 1000 \times 79/100)$		
	a =	$393 \times 415 / (0.85 \times 20 \times 1000) = 10$ mm			
	ϕ *Mn* =	ϕ *As fy* $(d - a/2)$ =	$0.9 \times 393 \times 415 \times (79 - 10/2) / 10^6 = 11$ kNm		
					> Mu; Safe. fos = 2.17
	Provide reinforcement at top and bottom faces:		10 mm diameter bars @	200 c/c.	

Ast provided = 393 mm^2 **Safe.**

Design of the side wall:

External face LF: 1.6

Load: Pwater + Psurch + Pearth + Pcable + topslab

Pwater =	2.81	×	1.6	=	4.50 kN
Psurch =	6.25	×	1.6	=	10 kN
Pearth =	5.63	×	1.6	=	9.01 kN
Wcable =	3.6	×	1.6	=	5.76 kN
UDL (factored)=			19.75 kN/m^2		

h1 = 0.25 m h2 = 0.63 m h3 = 0.42 m

Msidewallext = Pwater × h1 + Psurch × h2 + Pearth × h3 + Wcable × etray

Msidewallext = 4.5 × 0.25 + 10 × 0.63 + 9.01 × 0.42 + 5.76 x 0.4

= 14 kNm

Strength reduction factor, ϕ 0.9 ACI 318–19 (Table 21.2.1)

Mu **14.00** kNm

Required nominal strength, M_n = Mu/ϕ 15.56 kNm

(Continued)

TABLE 5.65 **(*Continued*)**
Design calculation

Trench mark	Calculation					
	$fc' =$	20	N/mm^2	$bw =$	1000	mm
	$fy =$	415	Mpa	$D =$	225	mm
				$d =$	179	mm
	Bar 5					

Bar 5 10 diameter @ 200 mm centers = 393 mm^2/m

Ast min = $0.18 \times 1000 \times 179/100 =$ 322 mm^2/m width

$a =$ $393 \times 415 / (0.85 \times 20 \times 1000) =$ 10 mm

$\phi\, Mn =$ $\phi\, As\, fy\, (d - a/2) =$ $0.9 \times 393 \times 415 \times (179 - 10 / 2) / 10^6 = 26$ kNm

> Mu; Safe.

Internal face:

Load: Pwater inside + no backfill (during the construction stage)

Moment inside the face of the wall = Pwater × h1

Msidewallint = $4.5 \times 0.25 =$ 1.13 kNm

Strength reduction factor,			ϕ	0.9		ACI 318–19 (Table 21.2.1)
			Mu	**1.13**	kNm	
Required nominal strength, M_n =			Mu/ϕ	1.26	kNm	
fc′ =	20	N/mm^2	bw =	1000	mm	
fy =	415	Mpa	D =	225	mm	
			d =	179	mm	

Bar 3

Let us try with 10 mm diameter bars@ 200 c/c. Ast = 393 mm^2

> Min reinforcement.

Ast min = 322 mm^2/m width (0.18 × 1000 × 179/100)

a = *393 × 415 / (0.85 × 20 × 1000) =* *10 mm*

ϕ Mn = *ϕ As fy (d – a/2) =* *0.9 × 393 × 415 × (179 – 10 / 2) / 10^6 =26 kNm*

> Mu; Safe.

Provide reinforcement
at top and bottom faces: 10 mm diameter bars @ 200 c/c.

Ast provided 393 mm^2 **Safe.**

Design of the base slab:

Load: Net base pressure upwards = 27.38 kN/m^2

(*Continued*)

TABLE 5.65 (*Continued*)
Design calculation

Trench mark	Calculation						
	Effective depth, d =			179	mm		
	Effective span =			1.38	m		
	Mspan =			5.21	kNm		
	Msupp =			4.35	kNm		
	Mu =	1.6 × 5.21 =	8	kNm		LF =	1.6
Bar 4							
Strength reduction factor,			ϕ	0.9		ACI 318–19 (Table 21.2.1)	
			Mu	**8.00**	kNm		
Required nominal strength, M_n =			Mu/ϕ	8.89	kNm		
fc′ =	20 N/mm²		bw =	1000	mm		
fy =	415 Mpa		D =	225	mm		
			d =	179	mm		
Bar 4							
let us try with		10 mm diameter bars@		200 c/c.	Ast = 393 mm²		
						> Min reinforcement.	
	Ast min = 322	mm²/m width	(0.18 × 1000 × 179/100)				

$a = \quad 393 \times 415 / (0.85 \times 20 \times 1000) = 10\ mm$

$\phi\ Mn = \quad \phi\ As\ fy\ (d - a/2) = 0.9 \times 393 \times 415 \times (179 - 10/2) / 10^6 = 26\ kNm$

> Mu; Safe. *3.19 fos*

Provide reinforcement
at top and bottom faces: 10 mm diameter bars @ 200 c/c.

Ast provided 393 mm^2 **Safe.**

End

5.11 DESIGN OF THE GROUND FLOOR SLAB

DESIGN OF THE GROUND FLOOR SLAB

The ground floor slab is a reinforced concrete slab resting on compacted backfill around foundations, trenches over the plinth area. These trenches, building foundation pits and equipment foundations are closely placed in the ground floor. So, it is difficult to use a large compactor machine and achieve the desired compaction.

Hence, it is advisable to use a thick layer of crushed stone, morrum or small sized gravel or stones mixed with sand or stone dust, which will provide a compacted subgrade below the ground floor slab.

For strength design, this slab can be considered as spanning over a soft spot of 3 M span and is reinforced at both layers. The grade of concrete shall not be less than M25 with a bottom cover of 75mm, if cast directly on compacted earth. Otherwise, this slab is cast on 50 mm thick levelling concrete underlain by compacted subgrade or rolled over soil. Polythene sheets or membrane is also laid on subgrade to prevent subsoil corrosion.

Paving in outdoor other than road, the slab in the car parking area, which is subject to uniform loading of 5 kN–30 kN per square meter, can be defined under this category.

An example of design of a ground floor slab is given below:

Design parameters

Grade of concrete: M25 fck = 25 N/mm^2

Reinforcement bar: high-yield strength deformed bars (HYSD); fy = 415 25 N/mm^2

Unit weight of concrete, γ conc = 24 kN/m^3

Concrete cover = 40 mm at the bottom and 20 mm at the top

Reinforcement bar diameter, ϕ = 8 mm

Ground floor slab thickness = 250 mm Floor finish with hardener = 40 mm

Design load intensity

Dead load, DL:	250 thick slab	250 × 24 /1000 = 6.00 kN/m^2
	Floor finishes	40 × 20 /1000 = 0.80 kN/m^2
		Total = 6.80 kN/m^2

Live load or imposed load, LL: 10 kN/m^2

Strength design in accordance with IS Code 456:2000 and by limit state

UDL = 6.8 + 10 = 16.80 kN/m^2 (DL + LL)

Span = 3 m

Design moment = 16.8 × 3^2 / 10 = 15.12 kNm

Let us try with a 250 mm thick slab

Width, B = 1000 mm   Depth, D = 250 mm   clear cover = 40 mm
fck = 25 MPa   fy = 415 MPa
Reinforcement bars:
Span: 8 mm diameter @ 150 mm c/c at the bottom (Ast = 335 mm^2 / m)
Support: 8 mm diameter @ 150 mm c/c at the bottom (Ast = 335 mm^2 / m)

Design check:
Moment, Mu = 1.5 × 15.12 = 22.68 kNm   Load factor = 1.5
d prov = 250−40 − 0.5 × 8 = 206 mm
M_U Limit = 146 kNm   Singly reinforced. M_ULimit/bd^2= 3.45
M_u / bd^2 = 22680000 / (1000 × 206^2) = 0.53
pt = 0.12 % minimum steel   (SP 16 Table 2)
Ast reqd. = 0.12 × 1000 × 206 / 100 = 247 mm^2/m width
Ast provided = 335 mm^2/m width at the bottom layer > Ast required; safe
Ast min = 247 mm^2/m width

Strength design in accordance with ACI 318–19 and by the LRFD method
Span = 3.0 m   UDL = 6.8 kN/m^2 DL   and 10 kN/m^2 LL
Load combination = 1.2 DL + 1.6 LL
Design UDL = 1.2 × 6.8 + 1.6 × 10 = 24.16 kN/m^2

Design moment = 24.16 × 3^2 /10 = 21.74 kNm

Refer to Figure 5.78 for stress-strain diagrams in a slab
From the above stress block, $C = T$
Or, $0.85fc'. a. bw = As.fy$
Or, $a = As.fy /(0.85 fc' bw)$
Mu = 21.74 kNm
Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)
Required nominal strength, M_n = Mu / ϕ = 24.16 kNm
fc′ = 20 N/mm^2 fy = 415 MPa
bw = 1000 m   D = 250 mm   d = 206 mm
Let us try with 10 mm diameter bars @ 200 mm c/c.   Ast = 393 mm^2 > min re-bars.
Ast min = 0.18 × 1000 × 206 / 100 = 371 mm^2 / m width.
a = 393 × 415 / (0.85 × 20 × 1000) = 10 mm
$\phi\, Mn = \phi\, As fy\, (d - a/2) = 0.9 \times 393 \times 415 \times (206-10/2)/10^6 = 30$ *kNm > Mu; safe.*
Provide reinforcement at top and bottom faces: 10 mm diameter bars @ 200 mm c/c.

5.12 DESIGN OF A CULVERT

The workout examples show how to design a standard RCC box culvert used for crossing storm drains or pipes below the road way. Design forces are calculated using frame constants provided in Reinforced Concrete Designers Handbook – Reynolds Steedman (10th Ed).

Sketch

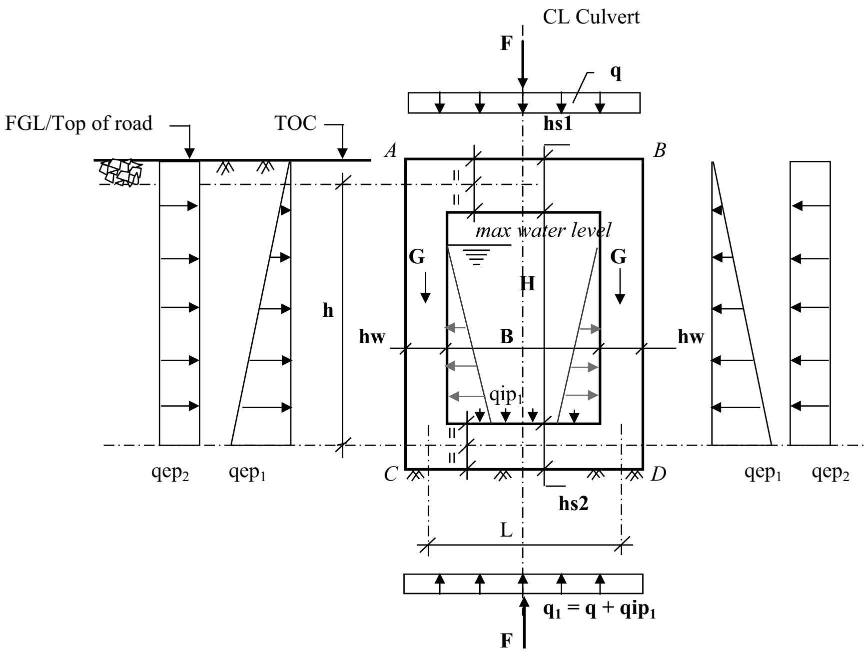

FIGURE 5.64 Sectional view and loading diagram

Design parameters

H = 2.5 m Clear height
B = 2 m Clear width
SBP = 150 kN/m^2 Safe bearing pressure
hs1 = 0.30 m Top slab thickness
hs2 = 0.25 m Base slab thickness
hw = 0.25 m Wall thickness
γsoil = 18 kN/m^3 Unit weight of soil
γconc = 25 kN/m^3 Unit weight of concrete
K_0 = 0.5 Coefficient of earth pressure at rest
γsurch = 40 kN/m^2 Ground surcharge for vehicular load
FGL = 0.0 M Finish-grade level/top of road
TOC = 0.0 M Top of a concrete culvert
Maximum water level in a culvert = 0.15 m below the soffit of the top slab
Grade of concrete = M25 fck = 25 Mpa
Reinforcing steel = Fe 415 fy = 415 MPa
Clear cover to reinforcement = 25 mm

Maximum bar diameter ϕ = 12 mm
The supporting ground of the culvert is medium-dense compacted soil.

Loads:

Wheel load on the top, F = 61 kN (1.1 × 110 /2) for axle load 110kN + 10% impact.
Uniform load on the top, q = 7.5 kN/m (0.3 × 25) self-weight of top slab and no earth cover.
Self-weight of a side wall, G = 15.63 kN/m (2.5 × 0.25 × 25)
Earth pressure on the wall, qep1 = 25.2 kN/m [0.5 × 18 × (2.5 + 0.3)]
Ground surcharge, qep2 = 20 kN/m (0.5 × 40)
Hydraulic pressure – internal, qip_1 = 23.5 kN/m [10 × (2.5 – 0.15)]

TABLE 5.66
Frame Constants

h	*2.8*	*k*	*1.24*	*K4*	*14*
L	*2.25*	*K1*	*2.24*	*K5*	*5.48*
hs	*0.3*	*K2*	*3.24*	*K6*	*7.24*
hw	*0.3*	*K3*	*4.24*	*K7*	*9.48*
				K8	11.72

$h = 2.5 + 0.5 \times 0.3 + 0.5 \times 0.25 = 2.8$ m
$L = 2 + 0.25 = 2.25$ m
$hs = 0.3 + 0.25 / 2 = 0.28$ m
$k = (h / L) \times (hs / hw)^3$

$K1 = k + 1$ $\quad K2 = k + 2$ $\quad K3 = k + 3$ $\quad K4 = 4k + 9$
$K5 = 2k + 3$ $\quad K6 = k + 6$ $\quad K7 = 2k + 7$ $\quad K8 = 3k + 8$

Force and moment diagram

[Refer to Reinforced Concrete Designers Handbook – Reynolds Steedman 10th Ed]

Load F

$q_l = F / (L + hw)$ $\quad M_A = (–)\, FLK4 / (24K1K3)$ $\quad M_C = (K6/K4)\, M_A$
$q_l = 61 / (2.25 + 0.3) = 23.92$ kN/m^2
$M_A = – 61 \times 2.25 \times 13.96 / (24 \times 2.24 \times 4.24) = (–)\, 8.41$ kNm
$M_C = (7.24 / 13.96) \times (– 8.41) = (–)\, 4.36$ kNm
$M_{AB} = (61 \times 2.25 / 4) – 0.5\,(8.41 + 4.36) = 27.93$ kNm span moment
$M_{CD} = 23.92 \times 2.25^2 / 24 = 5.05$ kNm span moment

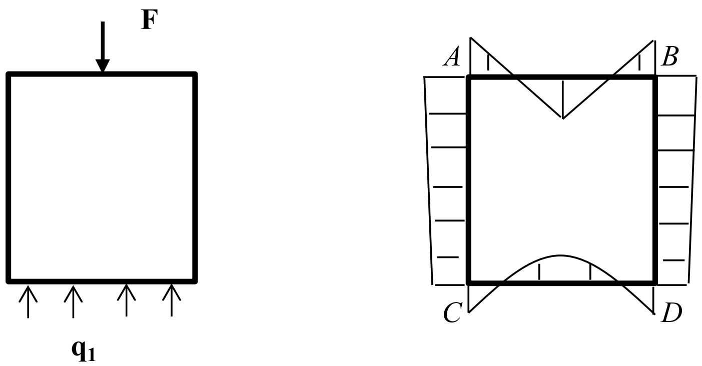

FIGURE 5.65 Force and moment diagram

Load q

$q_1 = q$ $M_A = M_C = (-)\ q\ L^2 / 12\ K1$
$q = 7.5$ kN/m
$M_A = -\ 7.5 \times 2.25^2 / (12 \times 2.24) = (-)\ 1.41$ kNm
$M_c = (-)\ 1.41$ kNm
$M_{AB} = (7.5 \times 2.25^2 / 8) - 1.41 = 3.34$ kN/m
$M_{CD} = 7.5 \times 2.25^2 / 24 = 1.58$ kNm

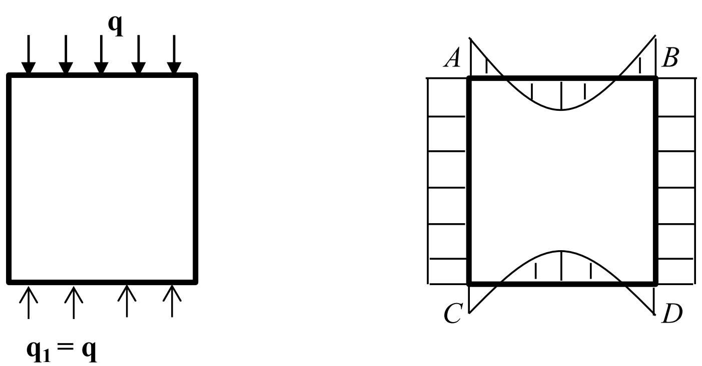

FIGURE 5.66 Force and moment diagram

Load G

$q_1 = 2G / (L + hw)$ $M_A = q_l\ L^2\ k / (12K1.K3)$ $M_C = (-)\ (K5.\ M_A) / k$
$q_l = 2 \times 15.63 / (2.25 + 0.3) = 12.26$ kN/m^2 per meter.
$M_A = 12.26 \times 2.25^2 \times 1.24 / (12 \times 2.24 \times 4.24) = 0.68$ kNm
$M_C = -\ (5.48 \times 0.68) / 1.24 = (-)\ 3.01$ kNm
$M_{AB} = 0.68$ kNm
$M_{CD} = 12.26 \times 2.25^2 / 24 = 2.59$ kNm

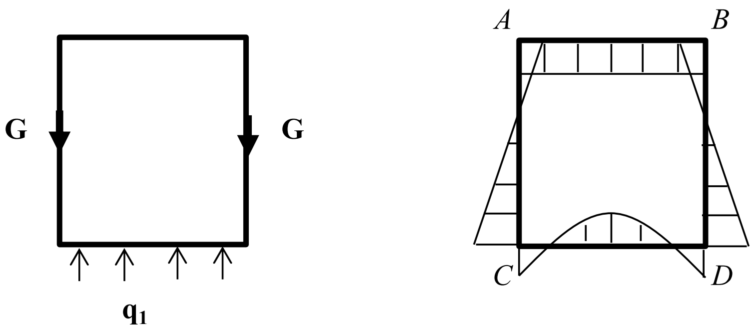

FIGURE 5.67 Force and moment diagram

Load q_{ep1}

$M_A = (-)\ qep_1\ h^2\ k\ K7 / (60\ K1\ K3)$ $\qquad$ $M_C = M_A\ (K8 / K7)$

$q_{ep1} = 25.2$ kN/m

$M_A = -\ 25.2 \times 2.8^2 \times 1.24 \times 9.48 / (60 \times 2.24 \times 4.24) = (-)\ 4.08$ kNm

$M_C = -\ 4.08 \times (11.72 / 9.48) = (-)\ 5.04$ kNm

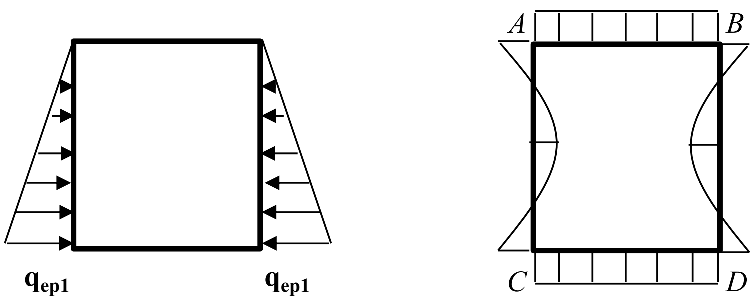

FIGURE 5.68 Force and moment diagram

Load q_{ep2}

$q_{ep2} = 20$ kN/m $\qquad$ $M_A = M_C = (-)\ qep_2\ h^2\ k / 12\ K1$

$M_A = M_C = -\ 20 \times 2.8^2 \times 1.24 / 12 \times 2.24 = (-)\ 7.23$ kNm

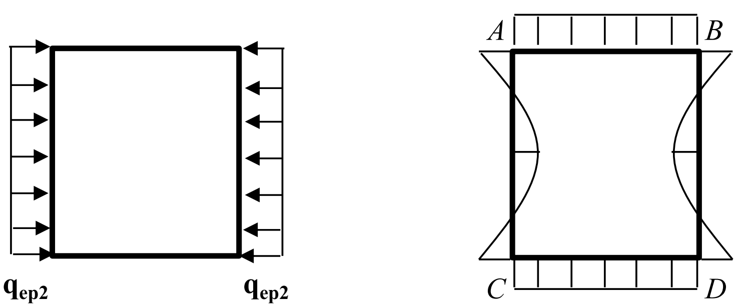

FIGURE 5.69 Force and moment diagram

Load q_{ip1}

$q_l = q_{ip1}$ $M_A = qip_l\ h^2\ k\ K7 / 60\ K1\ K3$ $M_C = M_A\ (K8 / K7)$

q_{lp1}= 23.5 kN/m

$M_A = 23.5 \times 2.8^2 \times 1.24 \times 9.48 / (60 \times 2.24 \times 4.24) = 3.8$ kNm

$M_C = 3.8 \times (11.72 / 9.48) = 4.7$ kNm

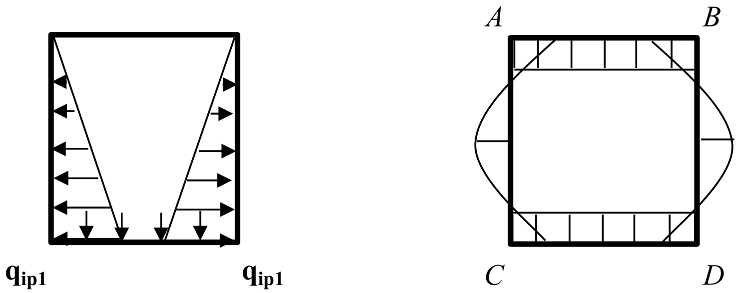

FIGURE 5.70 Force and moment diagram

TABLE 5.67
Member force

CASE I: OUTSIDE PRESSURE + INSIDE FULL UP TO THE MAXIMUM WATER LEVEL

			Base pressure	Support Moment		Span moment			
			q_1	M_A	M_C	M_{AB}	M_{CD}	M_{AC}	M_{BD}
Load type	**Load value**		**kN/m²**	**kNm**	**kNm**	**kNm**	**kNm**	**kNm**	**kNm**
F	61	kN	23.92	-8.41	-4.36	27.93	5.05	6.38	6.38
q	7.5	kN/m	7.50	-1.41	-1.41	3.34	1.58	1.41	1.41
G	15.63	kN	12.26	0.68	-3.01	0.68	2.59	1.16	1.16
qep1	25.2	kN/m	0.00	-4.08	-5.04	-4.08	5.04	4.56	4.56
qep2	20	kN/m	0.00	-7.23	-7.23	-7.23	7.23	3.62	3.62
qip1	23.5	kN/m	0.00	3.80	4.70	3.80	4.70	2.11	2.11
qip2	0	kN/m	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		Total	**43.68**	16.65	16.35	**24.44**	1.65	1.33	1.33

Sum base pressure, $\Sigma q_i <$ SBP; safe.

End shear:

Top slab $1 \times 61 + 0.5 \times 7.5 \times 2.25 = 71.5$ kN (wheel near end support)

Bottom slab $0.5 \times 43.68 \times 2.25 = 49$ kN

Side wall $0.67 \times (0.5 \times 25.2 \times 2.8) + 0.5 \times 20 \times 2.8 = 34$ kN (no water inside)

TABLE 5.68
Member force
CASE II: OUTSIDE PRESSURE + INSIDE EMPTY

			Base pr.	Support Moment		Span moment			
			q_1	M_A	M_C	M_{AB}	M_{CD}	M_{AC}	M_{BD}
Load type	**Load Value**		**kN/m²**	**kNm**	**kNm**	**kNm**	**kNm**	**kNm**	**kNm**
F	61	kN	23.92	-8.41	-4.36	27.93	5.05	-6.38	-6.38
q	7.5	kN/m	7.50	-1.41	-1.41	3.34	1.58	-1.41	-1.41
G	15.63	kN/m	12.26	0.68	-3.01	0.68	2.59	-1.16	-1.16
qep1	25.2	kN/m	0.00	-4.08	-5.04	-4.08	-5.04	4.56	4.56
qep2	20	kN/m	0.00	-7.23	-7.23	-7.23	-7.23	3.62	3.62
qip1	0	kN/m	0.00	0.00	0.00	0.00	0.00	0.00	0.00
qip2	0	kN/m	0.00	0.00	0.00	0.00	0.00	0.00	0.00
			43.68	-20.45	-21.05	20.64	-3.05	-0.77	-0.78

Dispersion of wheel load on the top slab

(Axle load 110 kN + 10% impact)

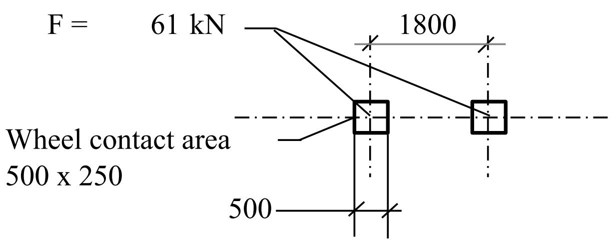

FIGURE 5.71 Wheel contact area

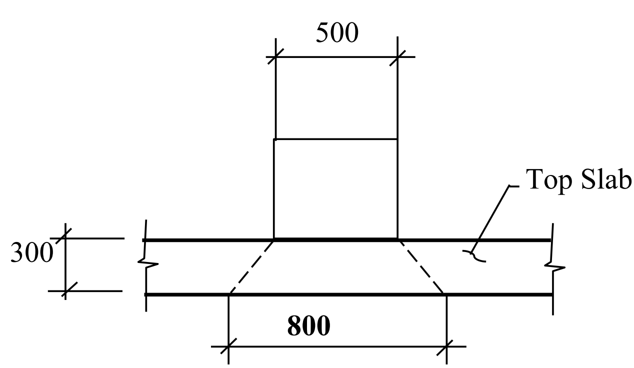

FIGURE 5.72 Wheel load dispersion

Design of member section

Strength design by IS code method – limit state *IS 456: 2000*

Top slab

Support moment = (–) 20.45 kNm (max) Span moment = 24.44 kNm (max)
End shear = 71.50 kN
Shear at the critical plane, i.e., d/2 away from the face of the wall
= 71.5 × (1.125 – 0.269) / (0.5 × 2.25)
= 54.4 kN
Slab thickness = 300 mm
Effective depth, deff = 300–25–0.5 × 12 = 269 mm
Effective width of wheel load dispersion, b = 800 mm

Check for moment:
Load factor = 1.7
$M_u^{span} / bd^2 = 1.7 \times 24.44 \times 10^6 / (800 \times 269^2) = 0.72$
$M_u^{support} / bd^2 = 1.7 \times 20.45 \times 10^6 / (800 \times 269^2) = 0.60$

At bottom layer: bars 2 and 3
pt = 0.22 %
Ast required = 0.216 × 1000 × 269 / 100 = 581 mm^2/m width
Ast provided = 377 + 377 = 754 mm^2/m width. safe.

At the top layer: bars 1 and 2
pt = 0.20 %
Ast required = 0.2 × 1000 × 269 / 100 = 538 mm^2/m width
Ast provided = 377 + 377 = 754 mm^2/m width. safe

Reinforcement [see Figure 5.73]

At span:
Bar mkd. 3 12 mm diameter @ 300 mm c/c (Ast = 377 mm^2 / meter width)
and
Bar mkd. 2 12 mm diameter @ 300 mm c/c (Ast = 377 mm^2 / meter width.)
At *support:*
Bar mkd. 1 12 mm diameter @ 300 mm c/c (Ast = 377 mm^2 / meter width)
and
Bar mkd. 2 12 mm diameter @ 300 mm c/c (Ast = 377 mm^2 / meter width)
Distribution bars: 12 mm diameter @ 200 mm c/c (Ast = 566 mm^2 / meter width)

Check for shear
Factored shear = 54.4 × 1.7 = 92 kN
$Ast_{provided}$ = 754 mm^2 / m pt = 0.35 % τc = 0.39 N/mm^2
$\tau c.b.d$ = 0.39 × 1000 × 269 = 105 kN > 92 kN. safe; shear reinforcement is not necessary.

Wall

Support moment = (–) 21.05 kNm (outer face) Span moment = 8.18 kNm (inner face) *
End shear = 34 kN
Shear at the critical plane, i.e., d/2 away from the face of support
= 34 × (1.4 – 0.219) / (0.5 × 2.8)
= 28.68 kN
Wall thickness = 250 mm
deff = 219 mm b = 1000 mm wall height, h = 2.8 m
(*) Max span moment without wheel load.
Load factor = 1.7
$M_u^{support} / bd^2 = 0.75$ $M_u^{span} / bd^2 = 0.29$

Outside: bar 5
pt = 0.22 %
Ast required = 473 mm^2/m width
Ast provided = 566 mm^2/m width. Safe.

Inside: bar 4
pt = 0.20 %
Ast required = 438 mm^2/m width
Ast provided = 566 mm^2/m width. Safe
Minimum reinforcement = 0.12 % per side

Reinforcement [see Figure 5.73]

Bar mkd. 5	12 mm diameter @ 200 mm c/c	(Ast = 566 mm^2 / meter width.)
Bar mkd. 4	12 mm diameter @ 200 mm c/c	(Ast = 566 mm^2 / meter width.)
Distribution bars:	12 mm diameter @ 200 mm c/c	(Ast = 566 mm^2 / meter width.)

Check for shear
Factored shear = 28.68 × 1.7 = 49 kN
$Ast_{provided}$ = 566 mm^2 / m pt = 0.26 % $\tau c = 0.36$ N/mm^2
τc.b.d = 0.36 × 1000 × 219 = 79 kN > 49 kN. safe; shear reinforcement is not necessary.

Base slab

Support moment = (–) 21.05 kNm Span moment = 5.05 +1.58 + 2.59 = 9.22 kNm
End shear = 49 kN
Shear at the critical plane, i.e., d/2 away from the face of support = 49 × (1.125 – 0.219) / (0.5 × 2.25) = 39 kN
Slab thickness = 250 mm deff = 219 mm b = 1000 mm
$M_u^{span} / bd^2 = 0.33$ $M_u^{support} / bd^2 = 0.75$ Load factor = 1.7

At top layer: Bar 6
pt = 0.20 %
Ast required = 438 mm^2/m width
Ast provided = 566 mm^2/m width. Safe.

At the bottom layer: bar 7
pt = 0.20 %
Ast required = 438 mm^2/m width
Ast provided = 566 mm^2/m width. Safe
Minimum reinforcement = 0.12 % per side

Reinforcement [see Figure 5.73]

Top – Bar mkd. 6	12 mm diameter @ 200 mm c/c	(Ast = 566 mm^2 / meter width)
Bottom – Bar mkd.7	12 mm diameter @ 200 mm c/c	(Ast = 566 mm^2 / meter width)
Distribution bars:	12 mm diameter @ 200 mm c/c	(Ast = 566 mm^2 / meter width)

Check for shear
Factored shear = 39 × 1.7 = 66 kN
$Ast_{provided}$ = 566 mm^2 / m pt = 0.26 % τc = 0.36 N/mm^2
$\tau c.b.d$ = 0.36 × 1000 × 219 = 79 kN > 66 kN. Safe; shear reinforcement is not necessary.

Detail of reinforcement placement

Concrete grade: M25 Reinforcement steel: HYSD [Fe 415]

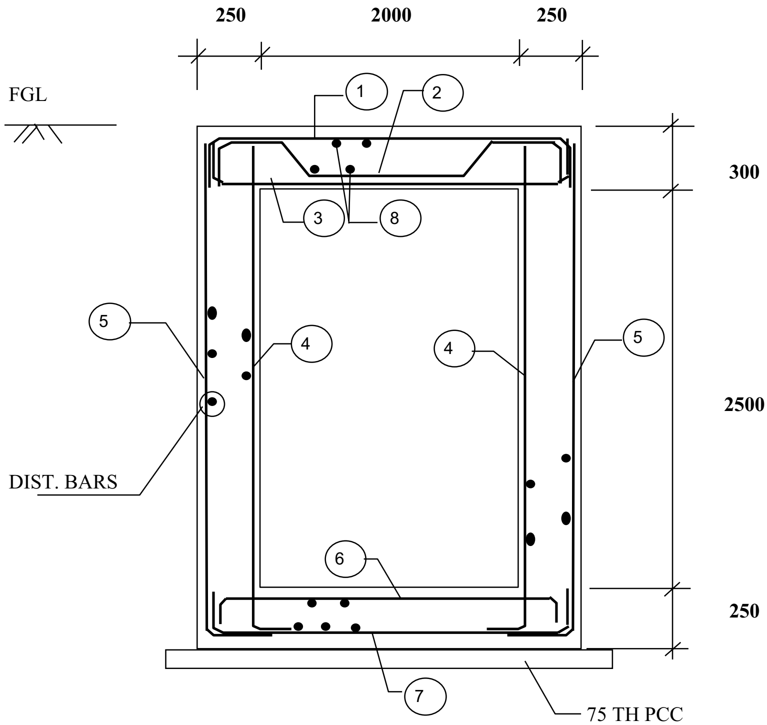

FIGURE 5.73 Reinforcement detail of the culvert

Bar mark	Description
1	12 mm diameter @ 300 mm C/C
2	12 mm diameter @ 300 mm C/C
3	12 mm diameter @ 300 mm C/C
4	12 mm diameter @ 200 mm C/C
5	12 mm diameter @ 200 mm C/C
6	12 mm diameter @ 200 mm C/C
7	12 mm diameter @ 200 mm C/C

Distribution bars:

8	12 mm diameter @ 200 mm C/C

Strength design by ACI Code – LRFD method ACI 318–19

Refer to Figure 5.78 for stress diagram, *C = T*
Or, *0.85fc′. a. bw = As.fy*
Or, *a = As.fy /(0.85 fc′ bw)*

Top slab

Span = 2.25 m Load combination = 1.2 DL + 1.6 LL
Moment span =1.2 × (3.34 + 0.68) + 1.6 × (27.93–4.08–7.23) = 31.42 kNm
Moment support = 1.2 × (–1.41 + 0.68) + 1.6 × (–8.41–4.08–7.23) = (–) 32.4 kNm

Span reinforcement:
Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)
Mu = 31.42 kNm Required nominal strength, M_n = Mu/ϕ = 34.91 kNm
fc′ = 20 N/mm^2 fy = 415 MPa
bw = 1000 m D = 300 mm d = 269 mm
Let us try with 12 mm diameter bars @ 150 mm c/c. Ast = 754 mm^2 > min re-bars.
Ast min = 0.18 × 1000 × 269 / 100 = 484 mm^2 / m width.
a = 754 × 415 / (0.85 × 20 × 1000) = 18 mm
$\phi\, Mn = \phi\, As\, fy\, (d - a/2) = 0.9 \times 754 \times 415 \times (269-18/2)/10^6 = 73$ *kNm > Mu; safe.*
Provide reinforcement bars 2 and 3,
12 mm diameter @ 150 mm c/c Ast provided = 754 mm^2

Support reinforcement:
Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)
Mu = –32.43 kNm Required nominal strength, M_n = Mu/ϕ = – 36.03 kNm
fc′ = 20 N/mm^2 fy = 415 MPa
bw = 1000 m D = 300 mm d = 269 mm
Let us try with 12 mm diameter bars @ 150 mm c/c. Ast = 754 mm^2 > min re-bars.
Ast min = 0.18 × 1000 × 269 / 100 = 484 mm^2 / m width.
a = 754 × 415 / (0.85 × 20 × 1000) = 18 mm
$\phi\, Mn = \phi\, As\, fy\, (d - a/2) = 0.9 \times 754 \times 415 \times (269-18/2)/10^6 = 73$ *kNm > Mu; safe.*
Provide reinforcement Bar – 1 and 2,
12 mm diameter @ 150 mm c/c Ast provided = 754 mm^2

Wall

Height = 2.80 m Load combination = 1.2 DL + 1.6 LL
Moment span =1.2 × (–1.41–1.16) + 1.6 × (– 6.38 + 4.56 + 3.62) = (–) 0.2 kNm
Moment support = 1.2 × (–1.41 + 0.68) + 1.6 × (–8.41–4.08–7.23) = (–) 32.4 kNm

Bar at the inner face: Bar 4
Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)
Mu = 0.2 kNm Required nominal strength, M_n = Mu/ϕ = 0.23 kNm
fc′ = 20 N/mm^2 fy = 415 MPa
bw = 1000 m D = 250 mm d = 219 mm
Let us try with 12 mm diameter bars @ 200 mm c/c. Ast = 566 mm^2 > min re-bars.
Ast min = 0.18 × 1000 × 219 / 100 = 394 mm^2 / m width.
a = 566 × 415 / (0.85 × 20 × 1000) = 14 mm
ϕ *Mn =*ϕ *As fy (d – a/2) = 0.9 × 566 × 415 × (219–14 / 2) /* 10^6 *= 45 kNm > Mu; safe.*
Provide reinforcement bar 4,
12 mm diameter @ 200 mm c/c Ast provided = 566 mm^2

Bar at the Outer face: bar 5
Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)
Mu = 32.43 kNm Required nominal strength, M_n = Mu/ϕ = 36.03 kNm
fc′ = 20 N/mm^2 fy = 415 MPa
bw = 1000 m D = 250 mm d = 219 mm
Let us try with 12 mm diameter bars @ 200 mm c/c. Ast = 566 mm^2 > min re-bars.
Ast min = 0.18 × 1000 × 219 / 100 = 394 mm^2 / m width.
a = 566 × 415 / (0.85 × 20 × 1000) = 14 mm
ϕ *Mn =*ϕ *As fy (d – a/2) = 0.9 × 566 × 415 × (219–14 / 2) /* 10^6 *= 45 kNm > Mu; safe.*
Provide reinforcement bar 5,
12 mm diameter @ 200 mm c/c Ast provided = 566 mm^2

Base slab

Span = 2.25 m Load combination = 1.2 DL + 1.6 LL
Moment span = 1.2 × (1.58 +2.59) + 1.6 × 5.05 = 13.08 kNm
Moment support = (–) 21.05 kNm

Span reinforcement: Bar 6 at the top layer
Strength reduction factor, ϕ = 0.9 for moment ACI 318–19 (Table 21.2.1)
Mu = 13.08 kNm Required nominal strength, M_n = Mu/ϕ = 14.54 kNm
fc′ = 20 N/mm^2 fy = 415 MPa
bw = 1000 m D = 250 mm d = 219 mm
Let us try with 12 mm diameter bars @ 200 mm c/c. Ast = 566 mm^2 > min re-bars.
Ast min = 0.18 × 1000 × 219 / 100 = 394 mm^2 / m width.
a = 566 × 415 / (0.85 × 20 × 1000) = 14 mm
ϕ *Mn =*ϕ *As fy (d – a/2) = 0.9 × 566 × 415 × (219–14 / 2) /* 10^6 *= 45 kNm > Mu; Safe.*
Provide reinforcement bar 6,

12 mm diameter @ 200 mm c/c Ast provided = 566 mm^2
Support reinforcement: Bar 7 at the bottom layer
Strength reduction factor, $\phi = 0.9$ for moment ACI 318–19 (Table 21.2.1)
Mu = 21.05 kNm Required nominal strength, $M_n = Mu/\phi = 23.39$ kNm
fc′ = 20 N/mm^2 fy = 415 MPa
bw = 1000 m D = 250 mm d = 219 mm
Let us try with 12 mm diameter bars @ 200 mm c/c. Ast = 566 mm^2 > min re-bars.
Ast min = 0.18 × 1000 × 219 / 100 = 394 mm^2 / m width.
$a = 566 \times 415 / (0.85 \times 20 \times 1000) = 14$ mm
$\phi\, Mn = \phi\, As\, fy\, (d - a/2) = 0.9 \times 566 \times 415 \times (219 - 14/2)/10^6 = 45$ *kNm > Mu; safe.*
Provide reinforcement bar 7,
12 mm diameter @ 200 mm c/c Ast provided = 566 mm^2

5.13 DESIGN CHARTS

TABLE 5.69
Limiting moment of resistance factor mu_{lim} / bd^2

	Mu_{lim} / bd^2		
fck N/mm^2	Fy =250 MPa	Fy = 415 MPa	Fy = 500 MPa
15	2.24	2.07	2.00
20	2.98	2.76	2.66
25	3.73	3.45	3.33
30	4.47	4.14	3.99

TABLE 5.70
Reinforcement bar details in us customary units / si unit

Size No #	C/section in^2	Wt lb/ft	Nom dia in	Perimeter in	Nom dia mm
3	0.11	0.38	0.38	1.18	10
4	0.2	0.67	0.50	1.57	12
5	0.31	1.04	0.63	1.96	16
6	0.44	1.50	0.75	2.36	20
7	0.6	2.04	0.88	2.75	22
8	0.79	2.67	1.00	3.14	25
9	1	3.40	1.13	3.54	28
10	1.27	4.30	1.27	3.99	32
11	1.56	5.31	1.41	4.43	36
14	2.25	7.65	1.69	5.32	43
18	4	13.60	2.26	7.09	60

TABLE 5.71
Design shear strength of concrete

100 As/bd pt %	M 20 τc	M 25 τc
<15	0.28	0.29
0.15	0.28	0.29
0.25	0.36	0.36
0.5	0.48	0.49
0.75	0.56	0.57
1	0.62	0.64

TABLE 5.72
Mu / Bd2 vs Percentage Reinforcement (pt and pc) for Flexural Member [d′/d = 0.10]

ƒck = 20 N/mm²					ƒck = 25 N/mm²				
	fy = 415 N/mm²		fy = 500 N/mm²			fy = 415 N/mm²		fy =500 N/mm²	
1	*2*	*3*	*4*	*5*	*1*	*2*	*3*	*4*	*5*
M_u/bd^2 N/mm²	pt %	pc %	pt %	pc%	M_u/bd^2 N/mm²	pt %	pc %	pt %	pc%
0	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
0.30	0.2	0.2	0.2	0.2	0.30	0.2	0.2	0.2	0.2
0.35	0.2	0.2	0.2	0.2	0.35	0.2	0.2	0.2	0.2
0.40	0.2	0.2	0.2	0.2	0.40	0.113	0.2	0.2	0.2
0.45	0.2	0.2	0.2	0.2	0.45	0.127	0.2	0.2	0.2
0.50	0.2	0.2	0.2	0.2	0.50	0.142	0.2	0.2	0.2
0.55	0.2	0.2	0.2	0.2	0.55	0.156	0.2	0.2	0.2
0.60	0.2	0.2	0.2	0.2	0.60	0.171	0.2	0.2	0.2
0.65	0.2	0.2	0.2	0.2	0.65	0.186	0.2	0.2	0.2
0.70	0.203	0.2	0.2	0.2	0.70	0.201	0.2	0.2	0.2
0.75	0.218	0.2	0.2	0.2	0.75	0.216	0.2	0.2	0.2
0.80	0.233	0.2	0.2	0.2	0.80	0.231	0.2	0.2	0.2
0.85	0.248	0.2	0.206	0.2	0.85	0.246	0.2	0.204	0.2
0.90	0.264	0.2	0.219	0.2	0.90	0.261	0.2	0.216	0.2
0.95	0.28	0.2	0.232	0.2	0.95	0.276	0.2	0.229	0.2
1.00	0.295	0.2	0.245	0.2	1.00	0.291	0.2	0.242	0.2
1.05	0.311	0.2	0.258	0.2	1.05	0.307	0.2	0.255	0.2
1.10	0.327	0.2	0.272	0.2	1.10	0.322	0.2	0.267	0.2
1.15	0.343	0.2	0.285	0.2	1.15	0.338	0.2	0.28	0.2
1.20	0.359	0.2	0.298	0.2	1.20	0.353	0.2	0.293	0.2
1.25	0.376	0.2	0.312	0.2	1.25	0.369	0.2	0.306	0.2

TABLE 5.72 (*Continued*)
Mu / Bd2 vs Percentage Reinforcement (pt and pc) for Flexural Member [d′/d = 0.10]

ƒck = 20 N/mm²					ƒck = 25 N/mm²				
	fy = 415 N/mm²		fy = 500 N/mm²			fy = 415 N/mm²		fy =500 N/mm²	
1	*2*	*3*	*4*	*5*	*1*	*2*	*3*	*4*	*5*
M_u/bd^2 N/mm²	pt %	pc %	pt %	pc%	M_u/bd^2 N/mm²	pt %	pc %	pt %	pc%
1.30	0.392	0.2	0.326	0.2	1.30	0.385	0.2	0.32	0.2
1.35	0.409	0.2	0.339	0.2	1.35	0.401	0.2	0.333	0.2
1.40	0.426	0.2	0.353	0.2	1.40	0.417	0.2	0.346	0.2
1.45	0.443	0.2	0.367	0.2	1.45	0.433	0.2	0.359	0.2
1.50	0.46	0.2	0.382	0.2	1.50	0.449	0.2	0.373	0.2
1.55	0.477	0.2	0.396	0.2	1.55	0.466	0.2	0.387	0.2
1.60	0.494	0.2	0.41	0.2	1.60	0.482	0.2	0.4	0.2
1.65	0.512	0.2	0.425	0.2	1.65	0.499	0.2	0.414	0.2
1.70	0.53	0.2	0.44	0.2	1.70	0.515	0.2	0.428	0.2
1.75	0.547	0.2	0.454	0.2	1.75	0.532	0.2	0.442	0.2
1.80	0.565	0.2	0.469	0.2	1.80	0.549	0.2	0.456	0.2
1.85	0.584	0.2	0.484	0.2	1.85	0.566	0.2	0.47	0.2
1.90	0.602	0.2	0.5	0.2	1.90	0.583	0.2	0.484	0.2
1.95	0.621	0.2	0.515	0.2	1.95	0.601	0.2	0.498	0.2
2.00	0.64	0.2	0.531	0.2	2.00	0.618	0.2	0.513	0.2
2.02	0.647	0.2	0.537	0.2	2.05	0.635	0.2	0.527	0.2
2.04	0.655	0.2	0.543	0.2	2.10	0.653	0.2	0.542	0.2
2.06	0.662	0.2	0.55	0.2	2.15	0.671	0.2	0.557	0.2
2.08	0.67	0.2	0.556	0.2	2.20	0.689	0.2	0.572	0.2
2.10	0.678	0.2	0.562	0.2	2.25	0.707	0.2	0.587	0.2
2.12	0.685	0.2	0.569	0.2	2.30	0.725	0.2	0.602	0.2
2.14	0.693	0.2	0.575	0.2	2.35	0.743	0.2	0.617	0.2
2.16	0.701	0.2	0.582	0.2	2.40	0.762	0.2	0.632	0.2
2.18	0.709	0.2	0.588	0.2	2.45	0.781	0.2	0.648	0.2
2.20	0.717	0.2	0.595	0.2	2.50	0.799	0.2	0.663	0.2
2.22	0.725	0.2	0.602	0.2	2.55	0.818	0.2	0.679	0.2
2.24	0.733	0.2	0.608	0.2	2.60	0.837	0.2	0.695	0.2
2.26	0.741	0.2	0.615	0.2	2.65	0.857	0.2	0.711	0.2
2.28	0.749	0.2	0.621	0.2	2.70	0.876	0.2	0.727	0.2
2.30	0.757	0.2	0.628	0.2	2.75	0.896	0.2	0.744	0.2
2.32	0.765	0.2	0.635	0.2	2.80	0.916	0.2	0.76	0.2
2.34	0.773	0.2	0.642	0.2	2.85	0.936	0.2	0.777	0.2
2.36	0.781	0.2	0.648	0.2	2.90	0.956	0.2	0.794	0.2
2.38	0.79	0.2	0.655	0.2	2.95	0.977	0.2	0.811	0.2
2.40	0.798	0.2	0.662	0.2	3.00	0.997	0.2	0.828	0.2

(*Continued*)

TABLE 5.72 (*Continued*)

Mu / Bd2 vs Percentage Reinforcement (pt and pc) for Flexural Member [d′/d = 0.10]

	fck = 20 N/mm²					fck = 25 N/mm²			
	fy = 415 N/mm²		fy = 500 N/mm²			fy = 415 N/mm²		fy =500 N/mm²	
1	*2*	*3*	*4*	*5*	*1*	*2*	*3*	*4*	*5*
M_u/bd^2 N/mm²	pt %	pc %	pt %	pc%	M_u/bd^2 N/mm²	pt %	pc %	pt %	pc%
2.42	0.806	0.2	0.669	0.2	3.05	1.018	0.2	0.845	0.2
2.44	0.814	0.2	0.676	0.2	3.10	1.039	0.2	0.863	0.2
2.46	0.823	0.2	0.683	0.2	3.15	1.061	0.2	0.88	0.2
2.48	0.831	0.2	0.69	0.2	3.20	1.082	0.2	0.898	0.2
2.50	0.84	0.2	0.697	0.2	3.25	1.104	0.2	0.916	0.2
2.52	0.848	0.2	0.704	0.2	3.30	1.126	0.2	0.935	0.2
2.54	0.857	0.2	0.711	0.2	3.32	1.135	0.2	0.942	0.2
2.56	0.866	0.2	0.719	0.2	3.34	1.144	0.2	0.945	0.2
2.58	0.874	0.2	0.726	0.2	3.36	1.153	0.2	0.951	0.2
2.60	0.883	0.2	0.733	0.2	3.38	1.162	0.2	0.957	0.2
2.62	0.892	0.2	0.74	0.2	3.40	1.171	0.2	0.963	0.2
2.64	0.901	0.2	0.748	0.2	3.42	1.18	0.2	0.968	0.2
2.66	0.91	0.2	0.755	0.2	3.44	1.189	0.2	0.973	0.2
2.68	0.919	0.2	0.758	0.2	3.46	1.197	0.2	0.979	0.2
2.70	0.928	0.2	0.765	0.2	3.50	1.21	0.2	0.989	0.2
2.77	0.958	0.2	0.783	0.2	3.60	1.24	0.2	1.014	0.2
2.8	0.968	0.012	0.791	0.2	3.70	1.271	0.2	1.04	0.2
2.9	0.998	0.045	0.816	0.2	3.80	1.302	0.2	1.065	0.2
3	1.029	0.077	0.842	0.2	3.90	1.333	0.2	1.091	0.2
3.1	1.06	0.109	0.868	0.2	4.00	1.363	0.2	1.116	0.2
3.2	1.091	0.142	0.893	0.2	4.10	1.394	0.211	1.142	0.215
3.3	1.122	0.174	0.919	0.2	4.20	1.425	0.244	1.167	0.242
3.4	1.152	0.207	0.944	0.204	4.30	1.456	0.276	1.193	0.27
3.5	1.183	0.239	0.97	0.232	4.40	1.487	0.309	1.219	0.298
3.6	1.214	0.271	0.995	0.259	4.50	1.517	0.341	1.244	0.326
3.7	1.245	0.304	1.021	0.287	4.60	1.548	0.374	1.27	0.353
3.8	1.276	0.336	1.046	0.314	4.70	1.579	0.407	1.295	0.381
3.9	1.306	0.369	1.072	0.342	4.80	1.61	0.439	1.321	0.409
4	1.337	0.401	1.098	0.369	4.90	1.641	0.472	1.346	0.437
4.1	1.368	0.433	1.123	0.397	5.00	1.671	0.504	1.372	0.464
4.2	1.399	0.466	1.149	0.425	5.10	1.702	0.537	1.397	0.492
4.3	1.429	0.498	1.174	0.452	5.20	1.733	0.57	1.423	0.52
4.4	1.46	0.53	1.2	0.48	5.30	1.764	0.602	1.449	0.548
4.5	1.491	0.563	1.225	0.507	5.40	1.795	0.635	1.474	0.575
4.6	1.522	0.595	1.251	0.535	5.50	1.825	0.667	1.5	0.603

TABLE 5.72 (*Continued*)

Mu / Bd2 vs Percentage Reinforcement (pt and pc) for Flexural Member [d′/d = 0.10]

ƒck = 20 N/mm²					ƒck = 25 N/mm²				
	fy = 415 N/mm²		fy = 500 N/mm²			fy = 415 N/mm²		fy =500 N/mm²	
1	*2*	*3*	*4*	*5*	*1*	*2*	*3*	*4*	*5*
M_u/bd^2 N/mm²	pt %	pc %	pt %	pc%	M_u/bd^2 N/mm²	pt %	pc %	pt %	pc%
4.7	1.553	0.628	1.276	0.563	5.60	1.856	0.7	1.525	0.631
4.8	1.583	0.66	1.302	0.59	5.70	1.887	0.733	1.551	0.659
4.9	1.614	0.692	1.328	0.618	5.80	1.918	0.765	1.576	0.686
5	1.645	0.725	1.353	0.645	5.90	1.948	0.798	1.602	0.714
5.1	1.676	0.757	1.379	0.673	6.00	1.979	0.83	1.627	0.742
5.2	1.707	0.79	1.404	0.701	6.10	2.01	0.863	1.653	0.77
5.3	1.737	0.822	1.43	0.728	6.20	2.041	0.896	1.679	0.797
5.4	1.768	0.854	1.455	0.756	6.30	2.072	0.928	1.704	0.825
5.5	1.799	0.887	1.481	0.783	6.40	2.102	0.961	1.73	0.853
5.6	1.83	0.919	1.506	0.811	6.50	2.133	0.993	1.755	0.881
5.7	1.861	0.952	1.532	0.839	6.60	2.164	1.026	1.781	0.908
5.8	1.891	0.984	1.558	0.866	6.70	2.195	1.059	1.806	0.936
5.9	1.922	1.016	1.583	0.894	6.80	2.226	1.091	1.832	0.964
6	1.953	1.049	1.609	0.921	6.90	2.256	1.124	1.857	0.992
6.1	1.984	1.081	1.634	0.949	7.00	2.287	1.157	1.883	1.019
6.2	2.014	1.114	1.66	0.976	7.10	2.318	1.189	1.909	1.047
6.3	2.045	1.146	1.685	1.004	7.20	2.349	1.222	1.934	1.075
6.4	2.076	1.178	1.711	1.032	7.30	2.38	1.254	1.96	1.103
6.5	2.107	1.211	1.736	1.059	7.40	2.41	1.287	1.985	1.13
6.6	2.138	1.243	1.762	1.087	7.50	2.441	1.32	2.011	1.158
6.7	2.168	1.276	1.788	1.114	7.60	2.472	1.352	2.036	1.186
6.8	2.199	1.308	1.813	1.142	7.70	2.503	1.385	2.062	1.213
6.9	2.23	1.34	1.839	1.17					
7	2.261	1.373	1.864	1.197					

Note: d’ = cover; d = effective depth
Minimum reinforcement percentage for beam = 0.2%; Min. Reinf. for slab = 0.12 %

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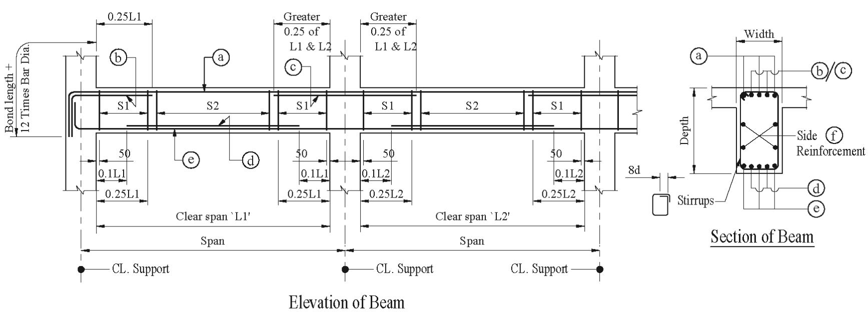
6 Construction details and reinforcement placement

This chapter includes standard detailing and reinforcement placement in reinforced cement concrete structures. These details are followed in present-day practices in the United States, India and Middle Eastern countries. The placement of reinforcement in beams, columns, slabs, staircases, footings, pile caps, walls and pre-stressed concrete piles; bending of bars at joints, different types of joints, drains, trenches and manholes, and culverts; mild steel embedment with lugs, edge protection angles, anchor bolts, grating and checkered plate floors; and protection of foundation concrete from subsoil corrosion have been provided in this chapter. The details are in conformity with American and British standards.



Schedule of Beam Reinforcement												
Beam Marked	T.O.C (M.)	Beam Size		Reinforcements						Stirrups		Remarks
		Width (mm)	Depth (mm)	Top			Bottom		Side (Each face)			
				a	b	c	d	e	f	S1	S2	

FIGURE 6.1A Beam Reinforcement – Sheet 1

DOI: 10.1201/9781003618119-6

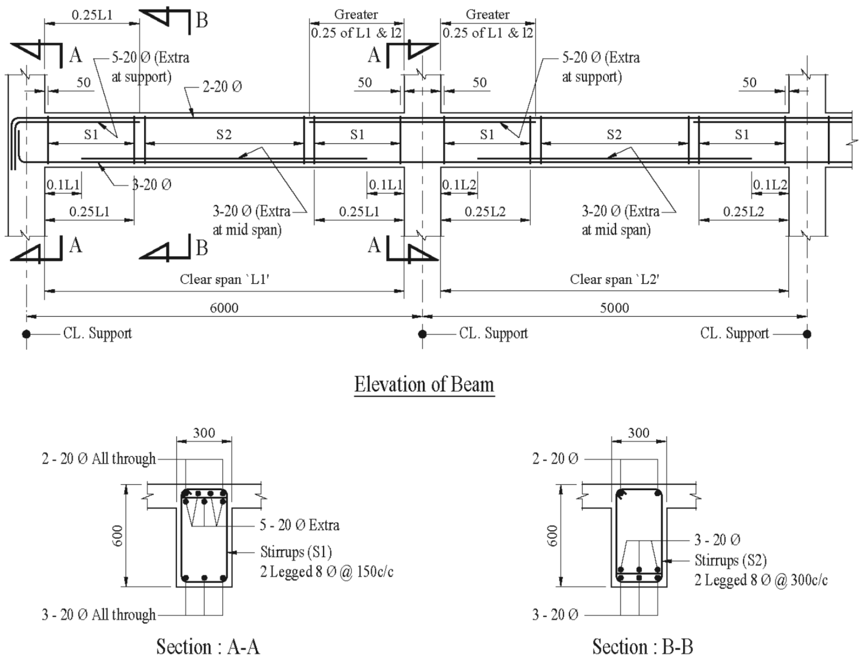


FIGURE 6.1B Beam Reinforcement – Sheet 2

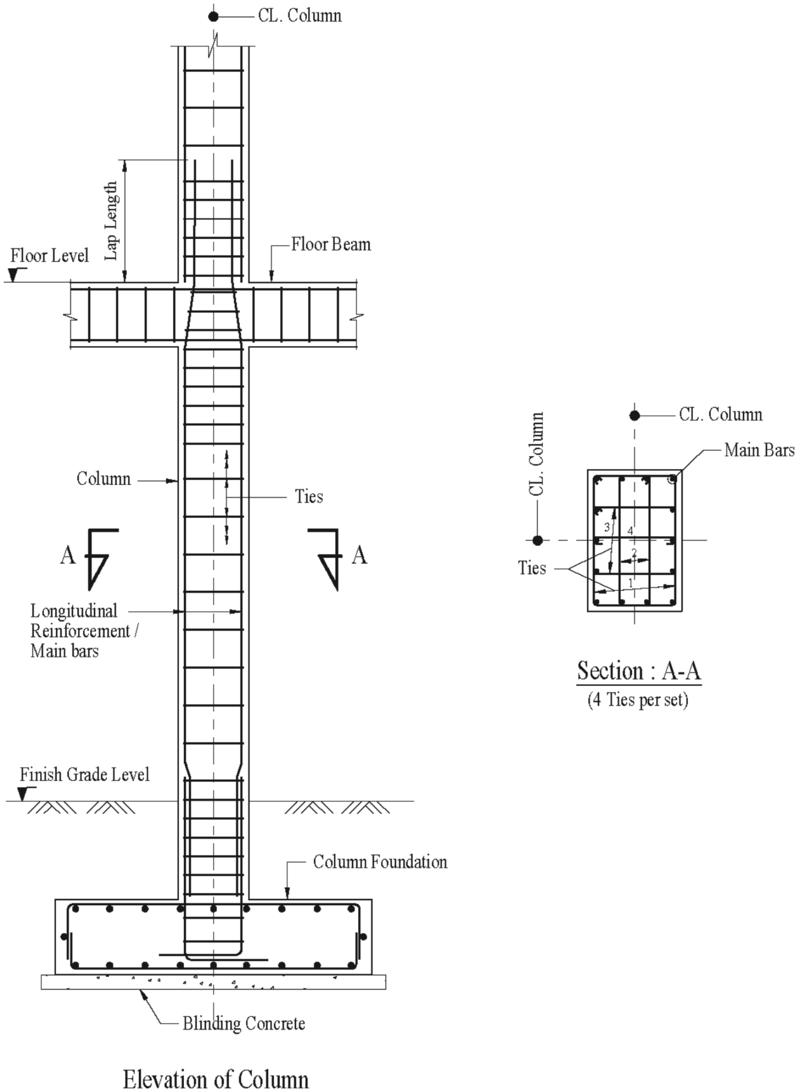


FIGURE 6.2 Column Reinforcement

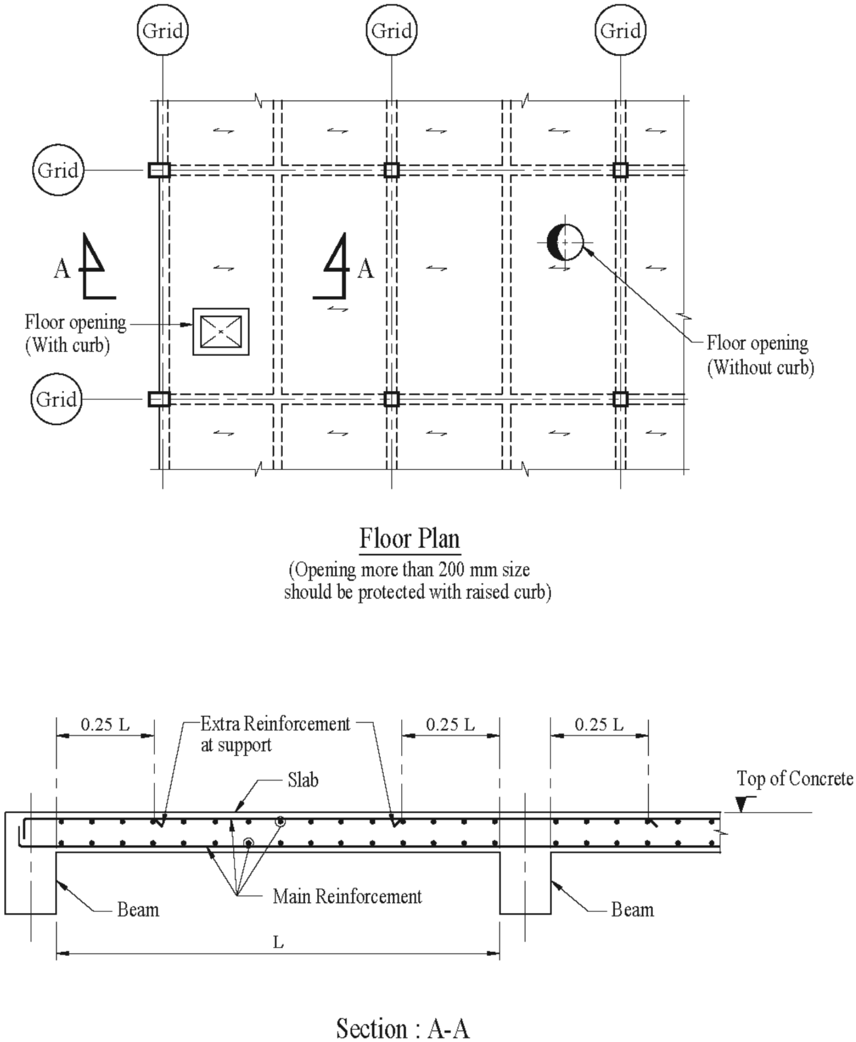


FIGURE 6.3A Slab Reinforcement – Plan and Section

(The slab reinforcement shown above is for industrial floor; for slabs in residential and lightly loaded floors, the bars in the top layer at mid span should not be necessary.)

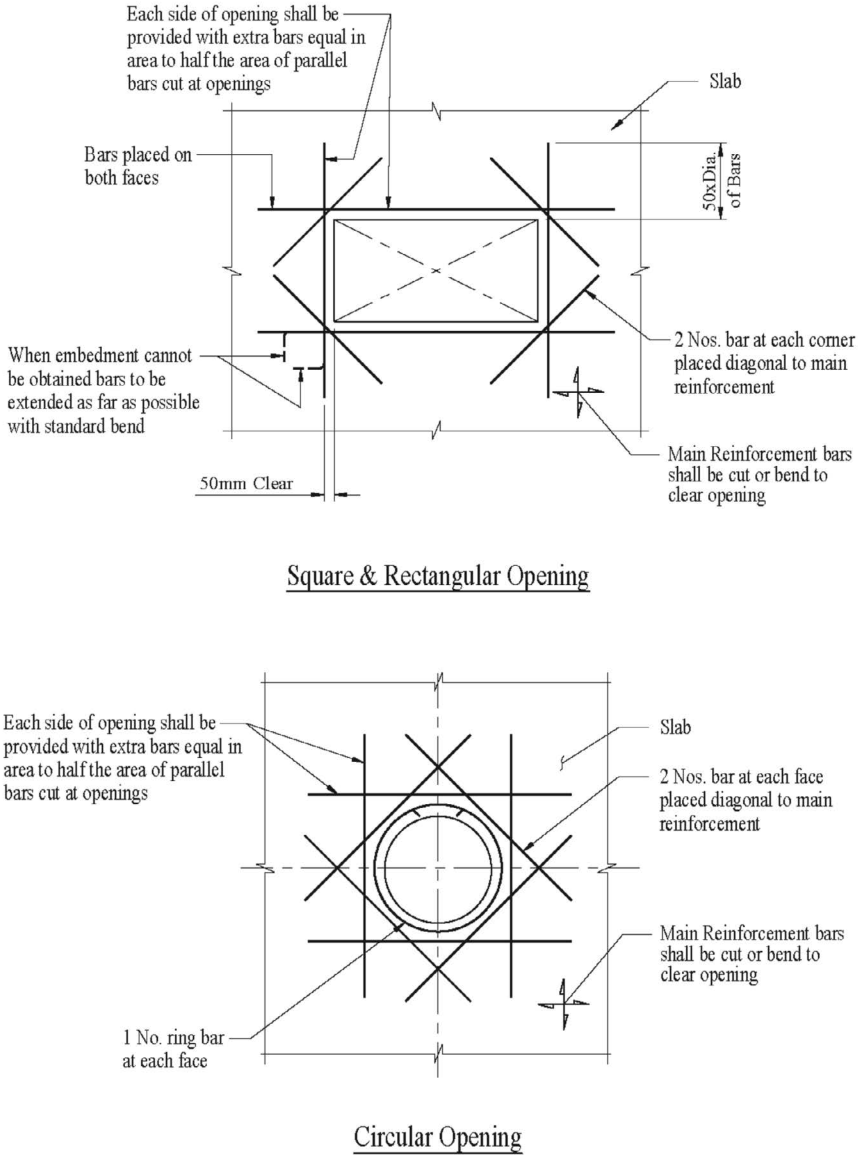


FIGURE 6.3B Slab Reinforcement around openings

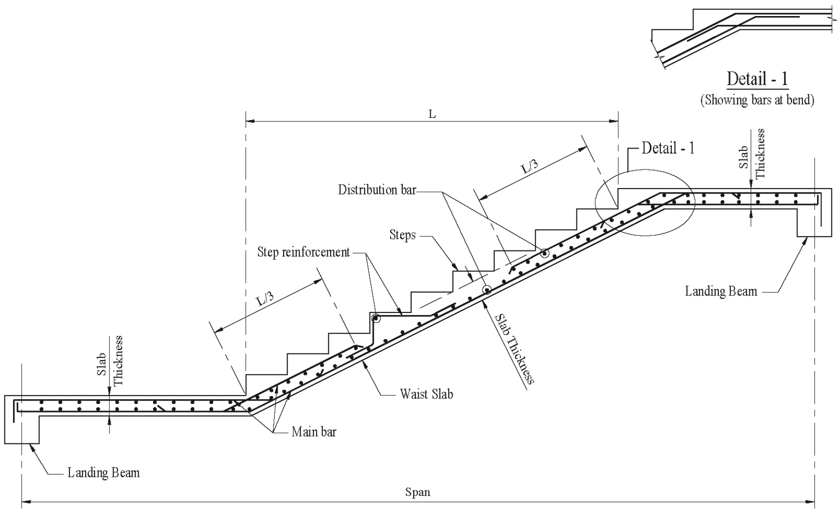


FIGURE 6.4 Stair case slab

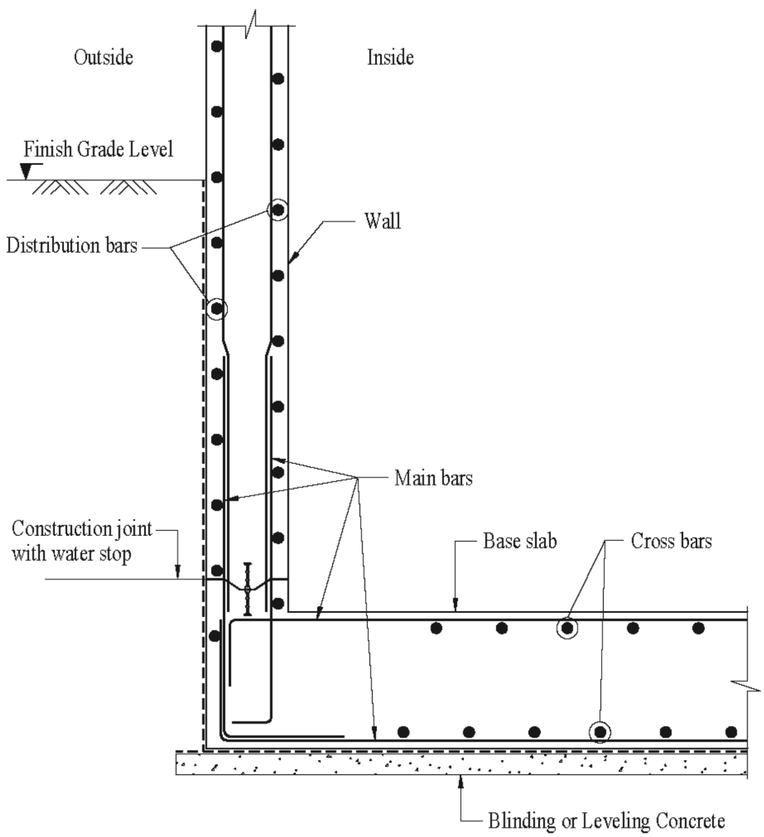
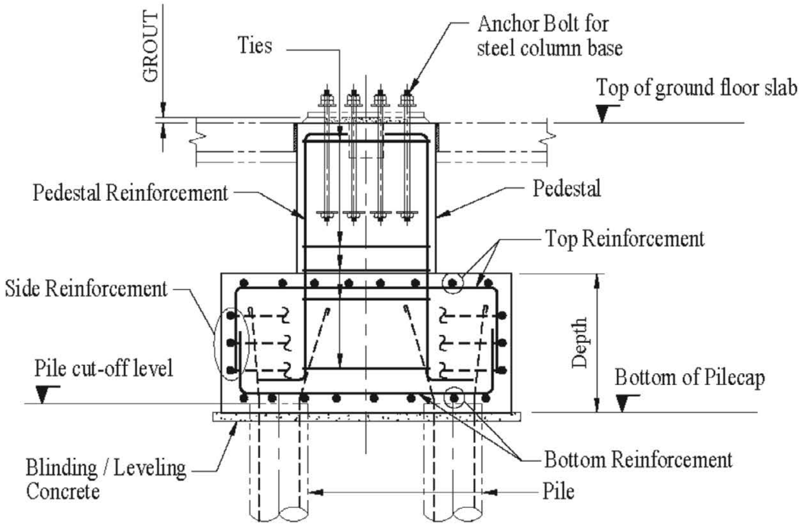
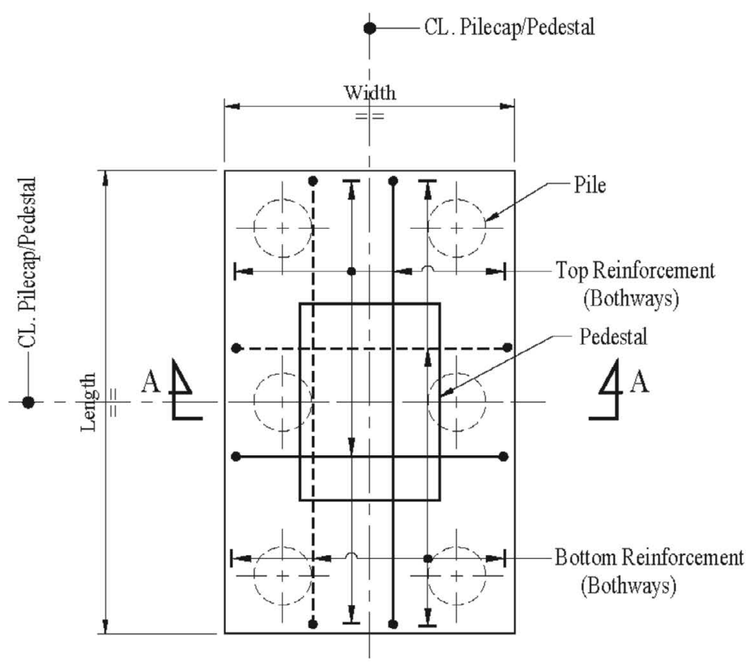


FIGURE 6.5 Wall reinforcement



Section : A-A



Plan

FIGURE 6.6 Pile cap reinforcement

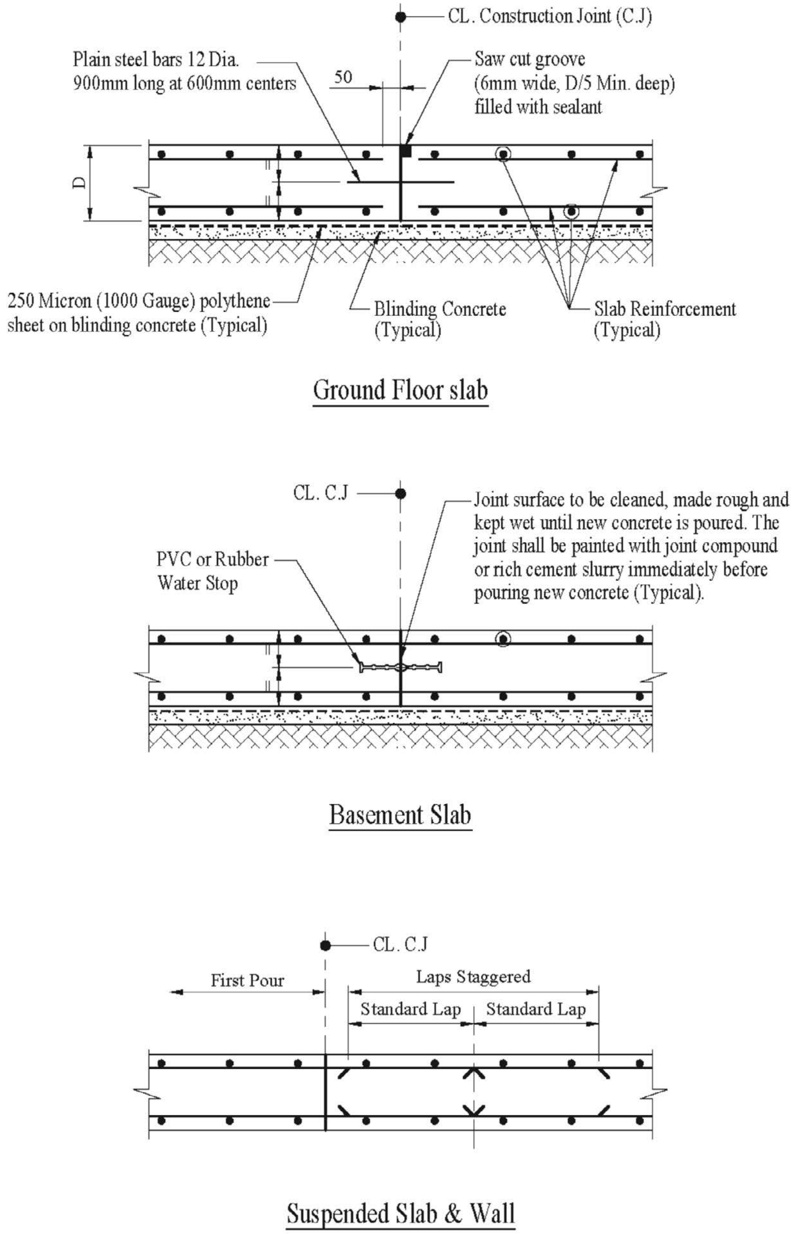


FIGURE 6.7 Construction Joints in Slab

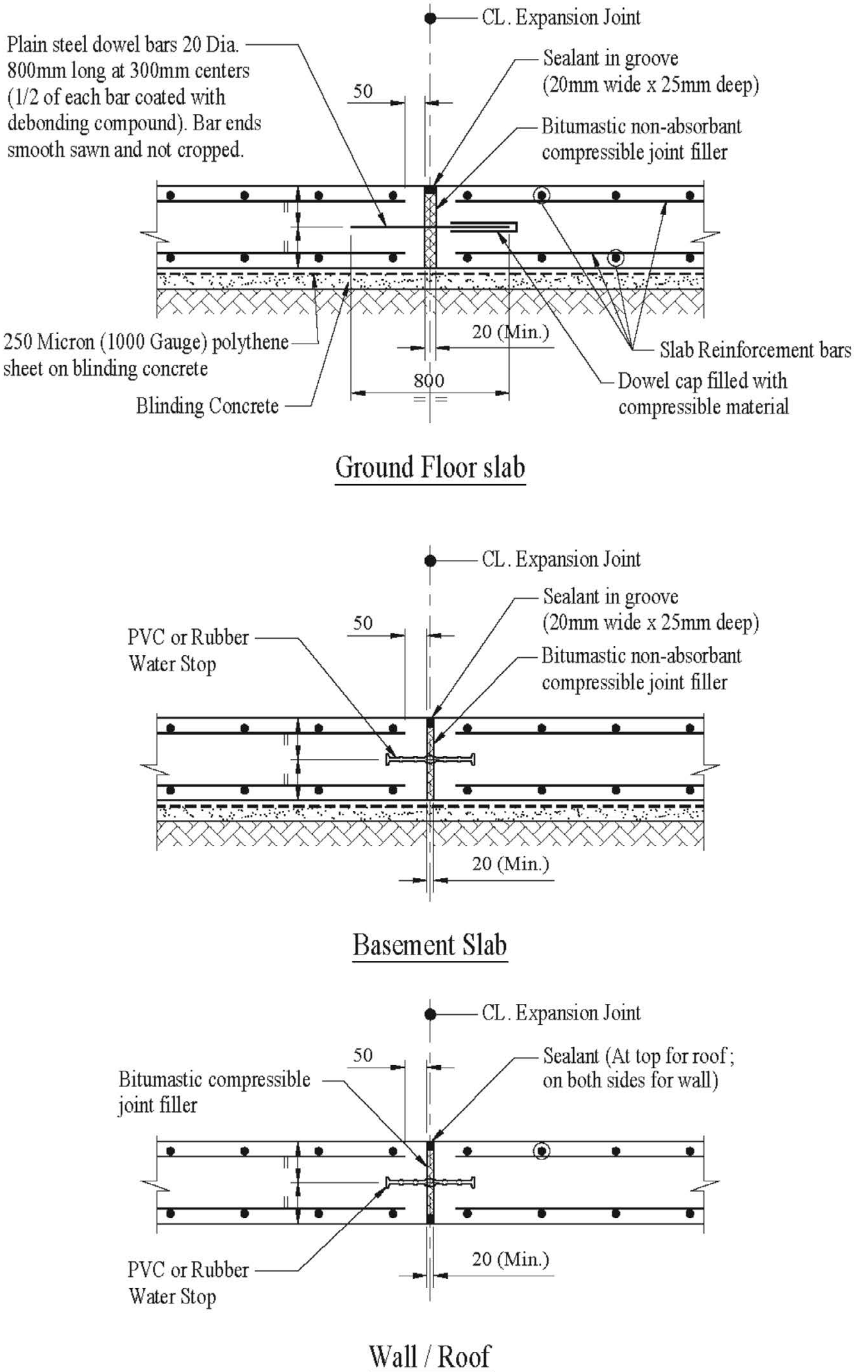
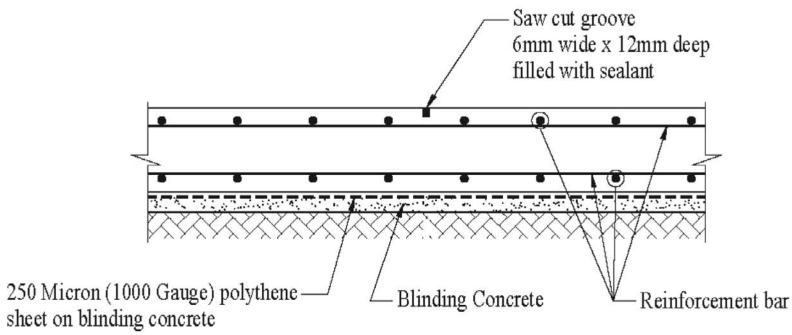
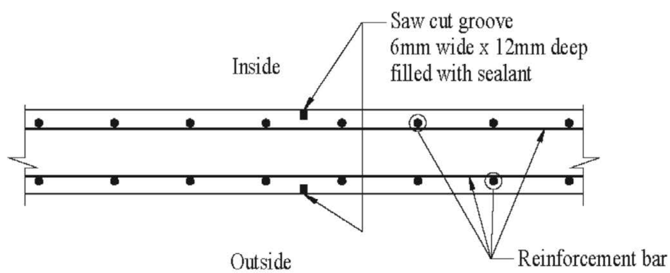


FIGURE 6.8 Expansion Joints



Ground Floor slab



Wall Structures

FIGURE 6.9 Crack Control Joints

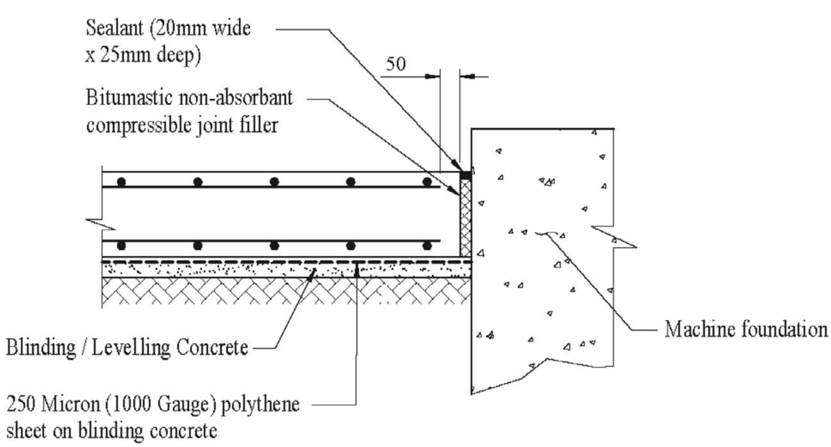


FIGURE 6.10 Separation Joints

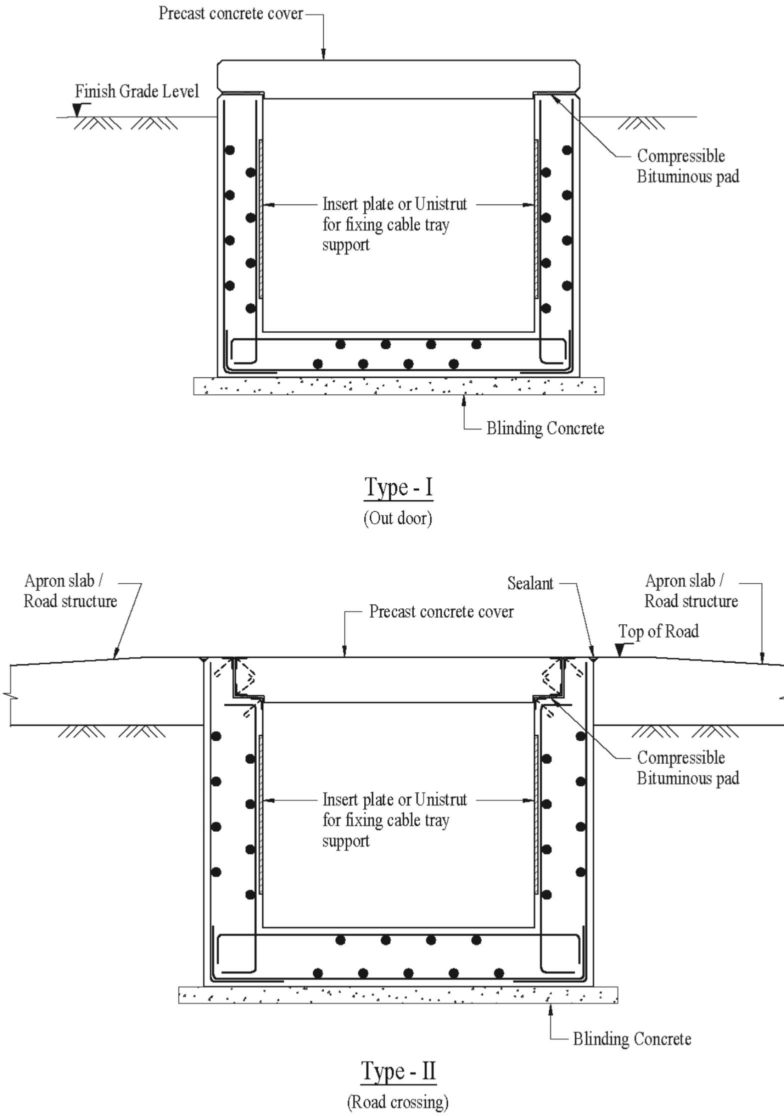


FIGURE 6.11 Cable Trench

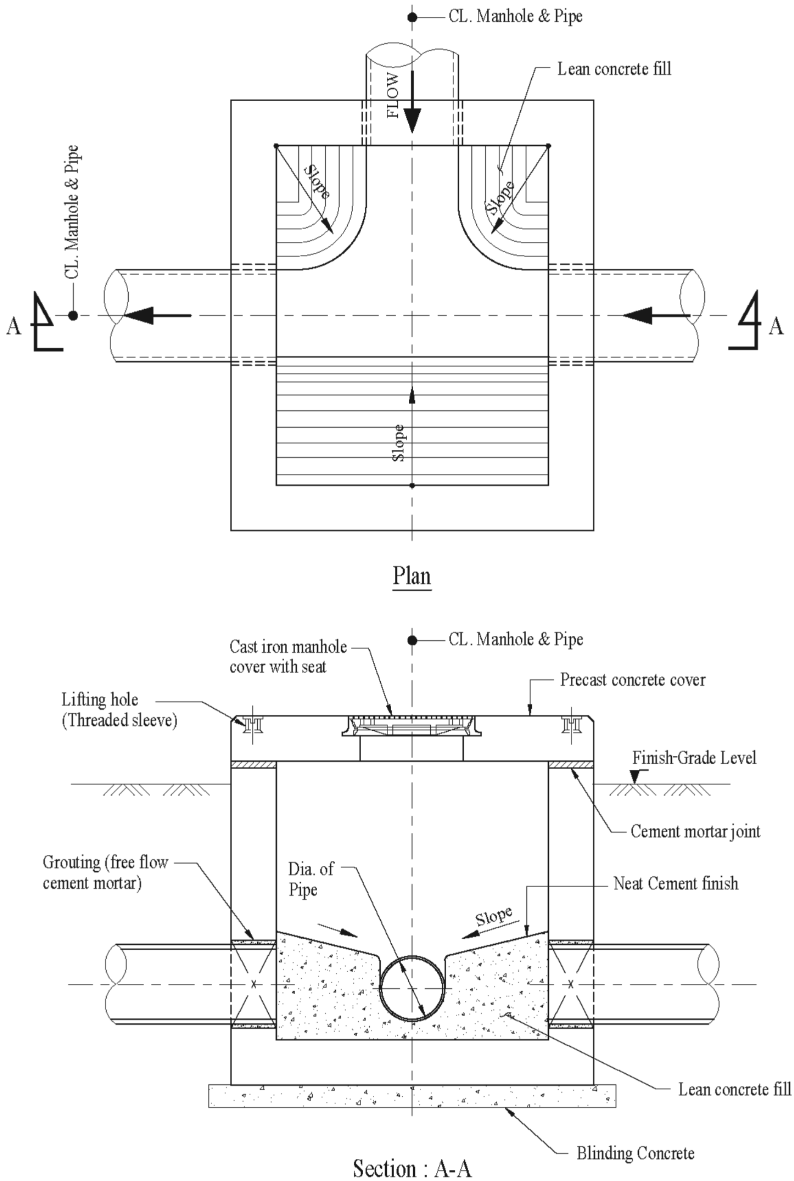


FIGURE 6.12A Manholes and Pits – Sheet 1

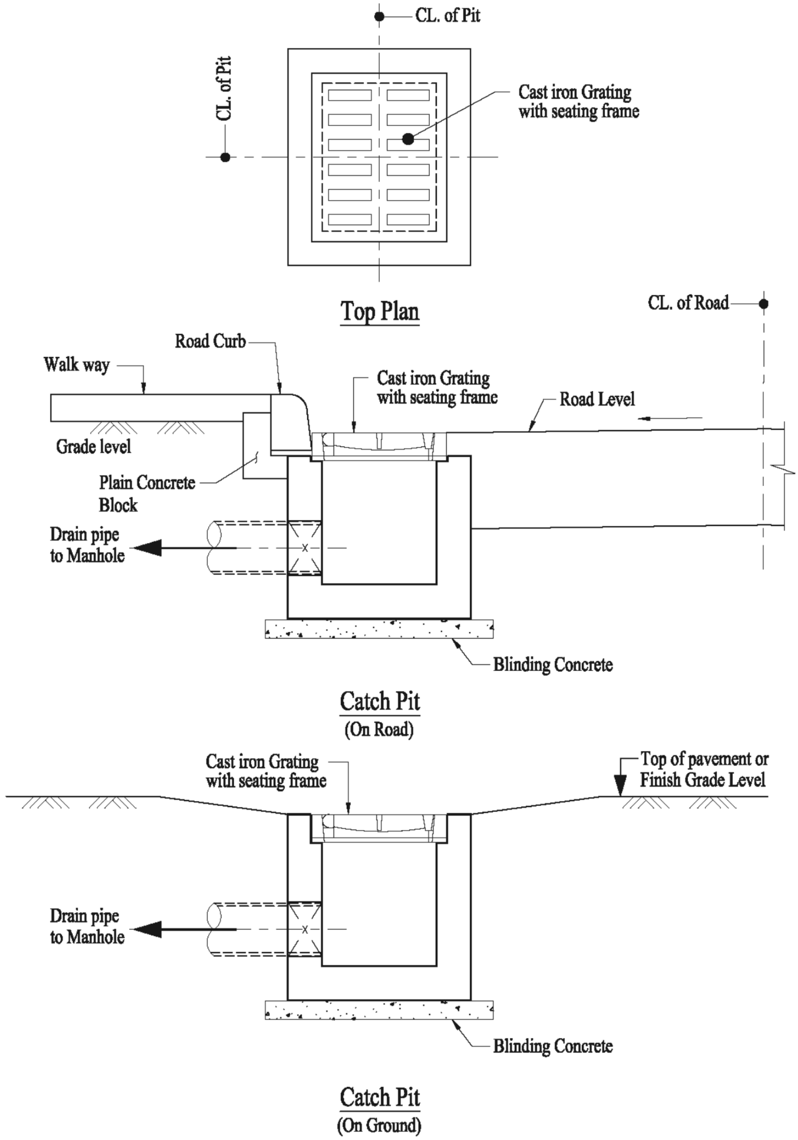


FIGURE 6.12B Manholes and Pits – Sheet 2

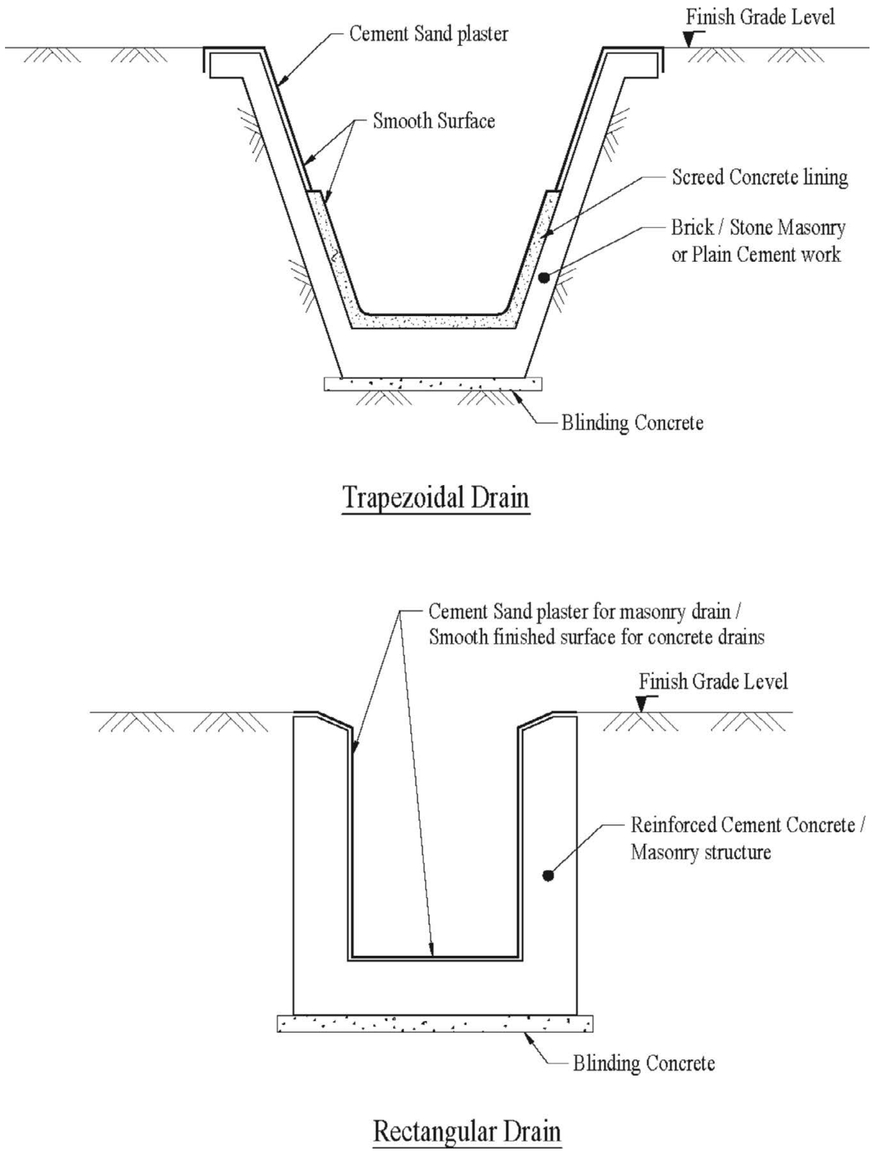
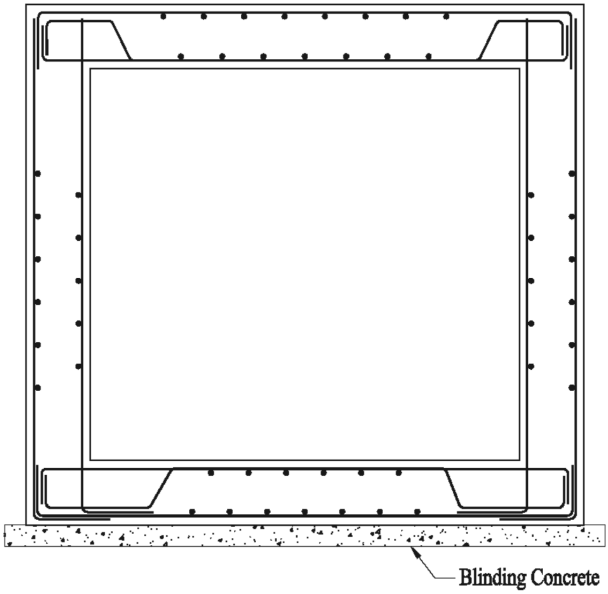
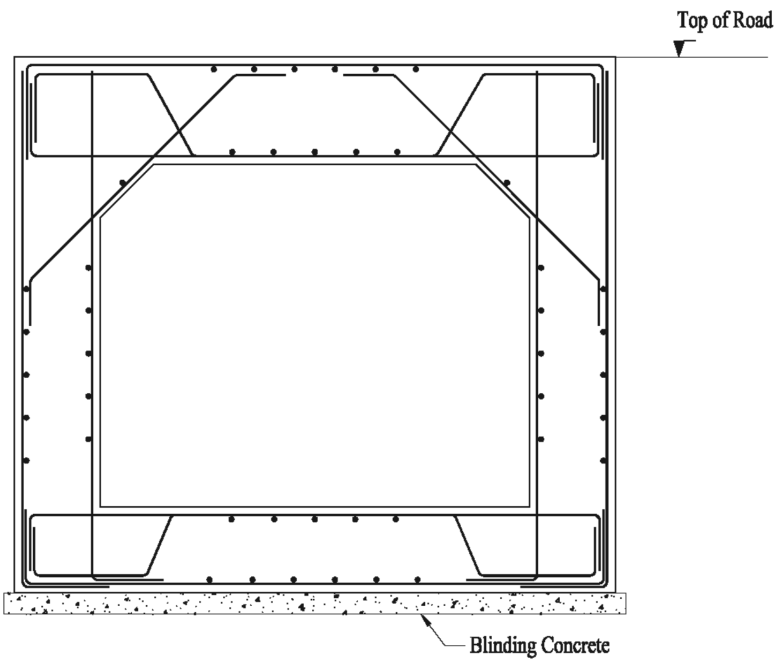


FIGURE 6.13 Surface Drains



Type - I
(Storm Drainage and Pipe Culvert)



Type - II
(Road Crossing Culvert)

FIGURE 6.14 Box Culvert

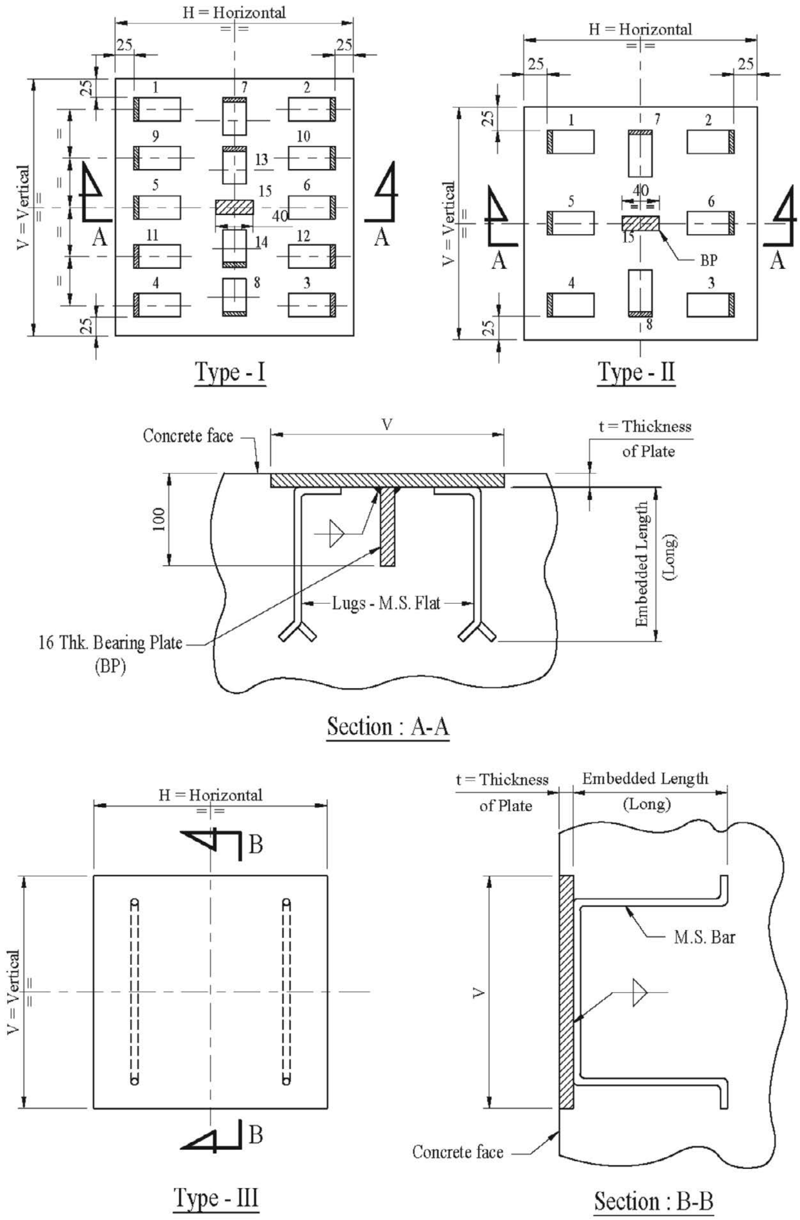


FIGURE 6.15A Embedded Plate – Sheet 1

Rectangular Plates

Item Marked	`V'x`H'	`t'	Lugs Type - Nos.	Location	Bearing Plate (BP)
EP-201	200 x 100	8	Round 12 - 6 Nos.	1 Thru. 6	Nil
EP-302	300 x 200	10	Flat 50 - 4 Nos.	1 Thru. 4 & 15	100 x 40 x 16 Thk.
EP-403	400 x 300	12	Flat 50 - 10 Nos.	1 Thru. 8, 13, 14	Nil
EP-504	500 x 400	12	Flat 50 - 14 Nos.	1 Thru. 14	100 x 40 x 16 Thk.
EP-604	600 x 400	16	Flat 50 - 14 Nos.		
EP-251	250 x 150	12	Flat 25 - 6 Nos.	1 Thru. 6	Nil
EP-252	250 x 200	12	Flat 50 - 6 Nos.	1 Thru. 4 & 15	100 x 40 x 16 Thk.
EP-352	350 x 250	12	Flat 50 - 6 Nos.	1 Thru. 6 & 15	
EP-452	450 x 250	12	Flat 50 - 10 Nos.	1 Thru. 6, 9 Thru. 12, & 15	
EP-652	650 x 250	16	Flat 50 - 14 Nos.		

Square Plates

Item Marked	`V'=`H'	`t'	Lugs Type - Nos.	Location	Bearing Plate (BP)
EP-100	100	8	Round 12 - 4 Nos.		Nil
EP-150	150	8	Round 12 - 4 Nos.		
EP-200	200	10	Flat 25 - 4 Nos.	1, 2, 3, 4	Nil
EP-250	250	12	Flat 25 - 8 Nos.	1 Thru. 8 & 15	100x40x 16 Thk.
EP-300	300	12	Flat 50 - 8 Nos.		
EP-400	400	12	Flat 50 - 10 Nos.	1, 2, 3, 4 5, 6, 7, 8, 13, 14	Nil
EP-450	450	12	Flat 50 - 10 Nos.		
EP-500	500	12	Flat 50 - 14 Nos.	1 Thru. 14	Nil
EP-600	600	16	Flat 50 - 14 Nos.		

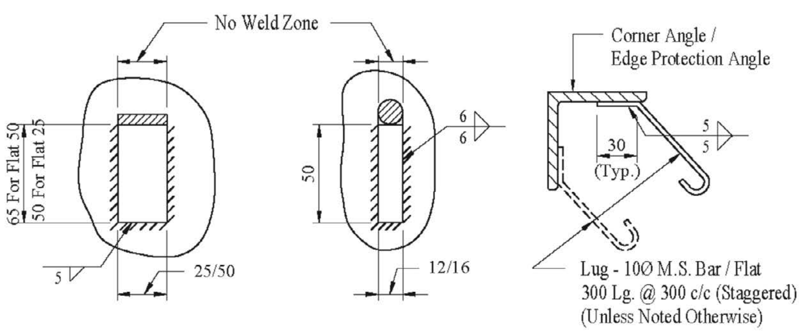
FIGURE 6.15B Embedded Plate – Sheet 2

Lugs for Embedded Plate

Type	Description
Flat - 25	25x6 Thk.x300 Lg.
Flat - 50	50x6 Thk.x450 Lg.
Round - 12	12Ø x200 Lg.
Round - 16	16Ø x300 Lg.

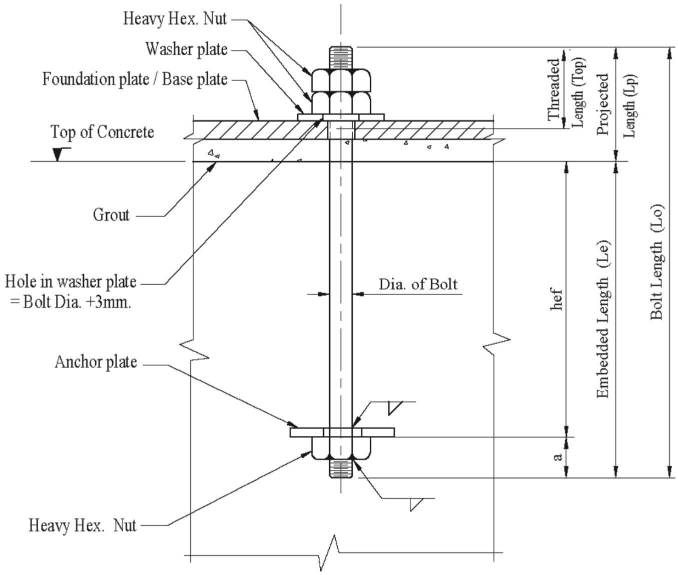
Corner Angle

Corner Angle Mark	Section Used	Lug
ISA - 45	ISA 45x45x5	10 Dia. Bar
ISA - 50	ISA 50x50x6	
ISA - 75	ISA 75x75x6	25x6 Thk. Flat
ISA - 100	ISA 100x100x8	



Welding of Lugs

FIGURE 6.15C Embedded Plate – Sheet 3



Schedule of Anchor Bolts												
Detail of Anchor Bolt								Anchor Plate		Washer Plate		Remarks
Dia. (mm)	Embedded Length (mm)			Projected Length (Lp)	Threaded Length (mm)		Total Length Lo (mm)	Dia. (mm)	Thickness (mm)	Dia. (mm)	Thickness (mm)	
	a	hef	Le		Top	Bottom						

FIGURE 6.16 Anchor bolts

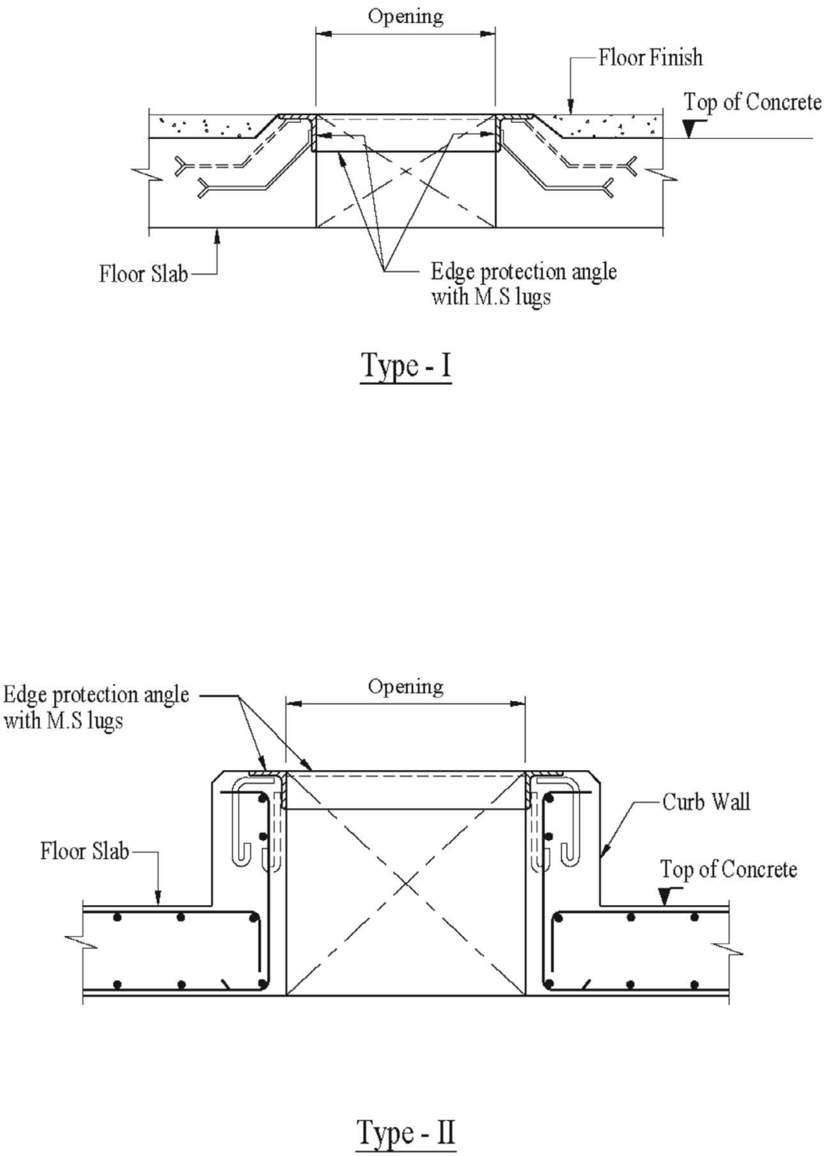


FIGURE 6.17 Edge protection detail around curb and opening

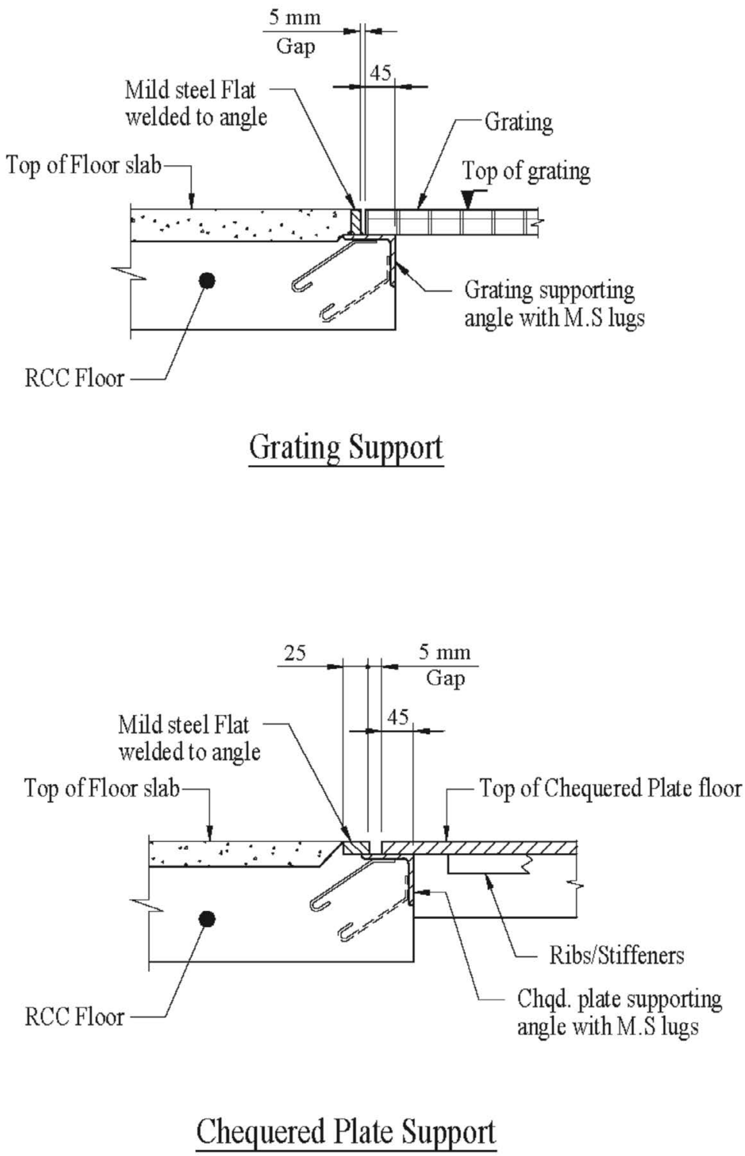


FIGURE 6.18 Grating and Chequered plate floor support

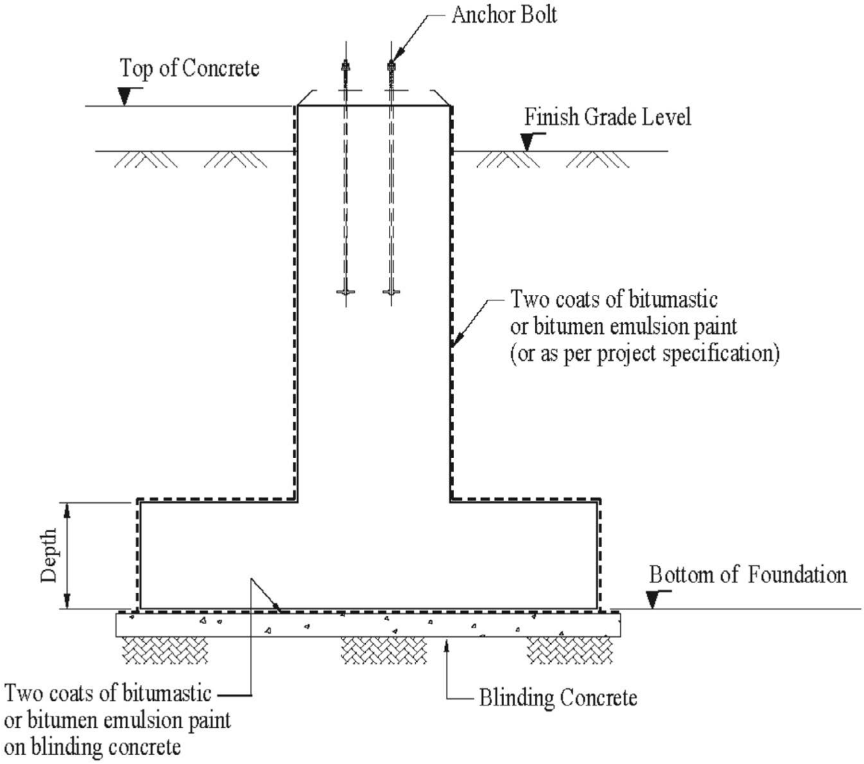


FIGURE 6.19 Protection of Foundation from Subsoil corrosion

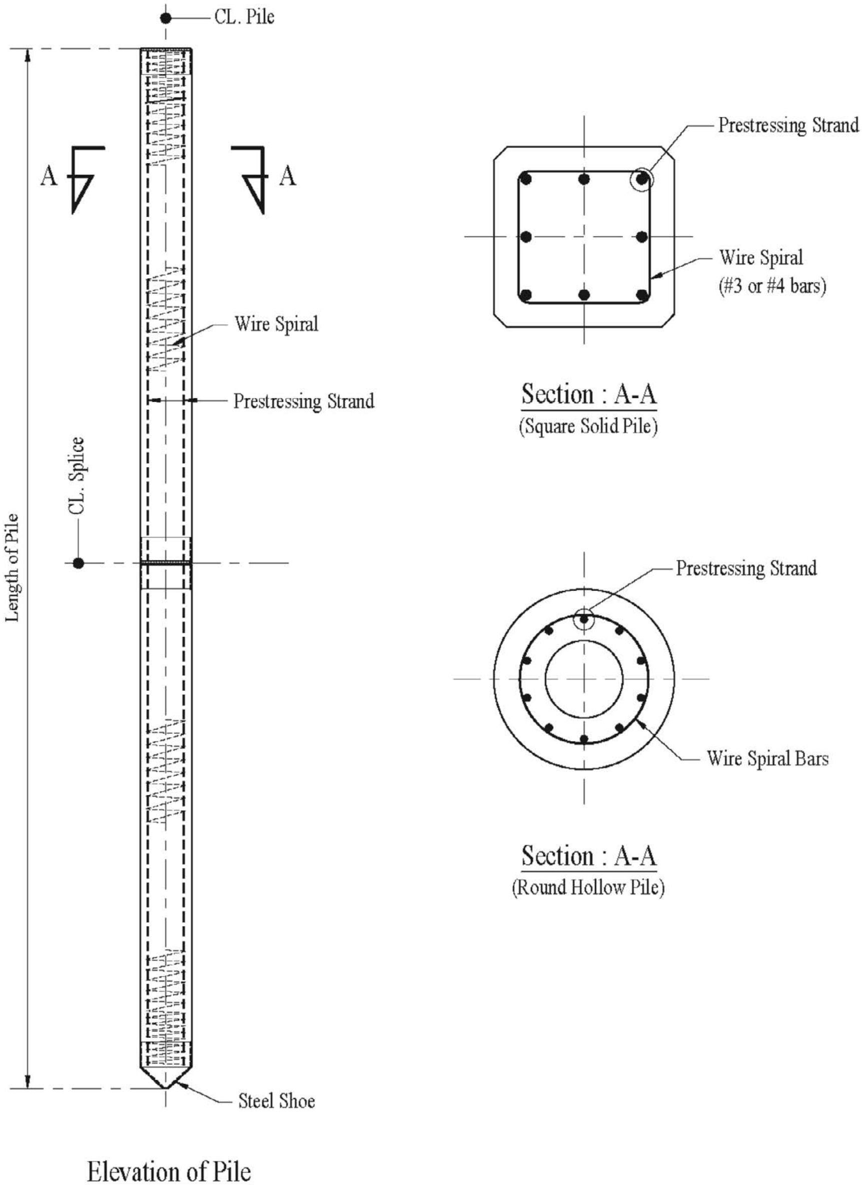


FIGURE 6.20 Pre-stressed Concrete Pile

REFERENCES

American Concrete Institute (ACI). *Building Code Requirements for Structural Concrete (ACI 318–19)*. USA: Farmington Hills, MI 48331, 19th ed., 2019.

Bureau of Indian Standards (BIS). *Indian Standard Plain and Reinforced Concrete – IS 456:2000*. New Delhi: 9 Bahadur Shah Zafar Marg, Pin 110002, 4th Revision, 2000.

7 Design of a large water reservoir on ground

7.1 DESCRIPTION OF STRUCTURE

The reservoir is a covered reinforced cement concrete structure resting on virgin soil or compacted ground. A partition wall divides the reservoir in two compartments. The size of each compartment is 40 M × 25 M clear in plan. The bottom of reservoir will be at one meter below the ground level. Minimum clear height available is four meters including free board. The roof slab will be designed as a flat slab supported by peripheral walls and rows of internal columns rising from the base raft. Peripheral walls are cantilever-type retaining walls with foundation cast monolithically with a base raft. All structures in contact with water will be designed as a uncracked section with reduced steel stress conforming to applicable codes and standards for water retaining structures.

7.2 REFERENCE CODES AND STANDARDS

a. Civil assignment Drawings: General arrangement and Plant drawing (Project specific drawings)
b. Codes and Standard: IS Codes: IS-3370, IS-456: 2000

7.3 MATERIALS USED

a. Levelling of screed concrete = M10 (cement:sand:coarse aggregate – 1:3:6)
b. Grade of cement concrete: M-25 (fck = 25 MPa)
c. Reinforcement bars: Fe 415 (yield strength not less 415 MPa)

7.4 DESIGN PARAMETERS

Finish-grade level (EL)	FGL = 1 M
Groundwater level (EL)	GWL = 0.5 M

Reservoir

Top of roof slab (EL)	TOP = 4.2 M	Roof slab thickness = 200 mm
Maximum water level (EL)	Maximum WL = 3.7 M	Screed on the raft = 125 mm (average thickness)
Free board	FB = 0.3 M	Raft thickness = 0.65 m
Minimum water level (EL)	MinWL = 1.1 M	Partition wall = 0.25 M at the top
Bottom of reservoir (EL)	BOT = 0 M	Periphery wall = 0.2 m at the top

DOI: 10.1201/9781003618119-7

Bottom of foundation (EL)	BOF = (–) 0.65 M
Length of reservoir (clear)	L = 40 m
Width of reservoir (clear)	B = 50 m
No. of compartments	NC = 2

Left compartment

Length	L_1 = 40 m
Width	B_1 = 25 m

No. of rows of internal columns along length	ncL = 10
No of rows of internal columns along width	ncB = 6
Spacing of internal columns along length	spcgL = 3.7 m
Spcg. of internal columns along width	spcgB = 3.6 m

Right compartment

Length	L_2 = 40 m
Width	B_2 = 25 m

No. of rows of internal columns along length	ncL = 10
No. of rows of internal columns along width	ncB = 6
Spacing of internal columns along length	spcgL = 3.7 m
Spacing of internal columns along width	spcgB = 3.6 m

Foundation soil

Gross bearing pressure	pgross = 340 kN/m^2	(for 25 mm settlement)
Shearing resistance	ϕ= 33 degree	
Cohesion	coh = 0 kg/cm^2	
Unit weight of soil backfill	γsoil = 18.5 kN/m^3	
Subgrade modulus	ksoil = 10.5 kg/cm^3	

Material

Concrete grade	M25	fck = 25 N/mm^2	Unit weight, γconc = 25 kN/m^3
Reinforcement steel	Fe 415	fy = 415 N/mm^2	(yield stress)

Load

Design wind pressure	pwind = 2.9 kN/m^2
Ground surcharge	γsurch = 10 kN/m^2

Load on roof

	Dead load (DL)		Live load (LL)	
RCC slab – 200 mm	5	kN/m^2		
Screed on roof – 50 mm average	1	kN/m^2		
Waterproofing coats	0.12	kN/m^2		
Ceiling plaster	0			
Total =	6.12	kN/m^2	1.5	kN/m^2

Rigidity of base slab

[Refer to cl.no. 5.1.1 and Appendix-C of IS: 2950]
To determine relative stiffness factor **K** of the foundation raft (> 0.5).
EI = Ec × I cm^4 Concrete grade M25 fck = 25 MPa
Length, a_{raft} = 4000 cm Width, b_{raft} = 2500 cm D_{raft} = 60 cm
$I_{raft} = 72 \times 10\ cm^4$
Ec = 285000 kg/cm^2
Modulus of compressibility of soil, Es = 500 kg/cm^2

For rectangular raft

$K = (Ec / 12Es) \times (Draft / b_{raft})^3 = 0.00065664$

For the whole structure

$K = (Ec \times Iraft) / (Es \times braft^3 \times a_{raft}) = 0.00065664$ [Cl.no 5.1.1(a) of IS: 2950]

Observation: K<0.5; check column spacing (less than 1.75/l) to satisfy the conventional method of design based on the assumption of linear distribution of contact pressure.

To determine critical column spacing (less than 1.75 / *l*)

Trial – 1

B = width of the raft in cm = 4000 cm
k = subgrade modulus in kg/cm^3 = 10.5 kg / cm^3
Ec = modulus of elasticity of concrete in kg/cm^3 = 285000 kg / cm^3
Draft = thickness of the raft in cm = 60 cm
I = moment of inertia of the raft in cm^4 = 72000000 cm^4

$$l = \sqrt[4]{\frac{kB}{4Ec\,I}} = 0.0048$$

Critical column spacing = 1.75 / *l* = 367cm = 3.67 m

Trial – 2

B = width of the raft in cm = 4000 cm
k = subgrade modulus in kg/cm^3 = 10.5 kg / cm^3
Ec = modulus of elasticity of concrete in kg/cm^3 = 285000 kg / cm^3
Draft = thickness of the raft in cm = 65 cm
I = moment of inertia of the raft in cm^4 = 91541667 cm^4

$$l = \sqrt[4]{\frac{kB}{4Ec\,I}} = 0.0040$$

Critical column spacing = 1.75 / *l* = 1.75 / 0.004 = 437 cm = 4.37 m
So, provide slab thickness = 0.65 m.

Rankin's analysis for minimum depth of foundation below ground level

Maximum base pressure developed, q = 57.57 kN/m^2 [from Section 7.8, gross pressure on soil]

Unit weight of soil, γsoil = 18 kN/m^3 Shearing resistance, ϕ = 34 degree

Minimum required depth, Dmin =

$$\frac{q}{\gamma soil}\left\{\frac{1-\sin f}{1+\sin f}\right\}^2$$

$$= \frac{111}{18}\left\{\frac{1-\sin 34^0}{1+\sin 34}\right\}^2$$

$$= 0.5\,\text{m}$$

Depth provided = 1−(−0.65) = 1.65 m > D min; Hence Safe.

7.5 SKETCHES

This section includes design sketches of the water reservoir.

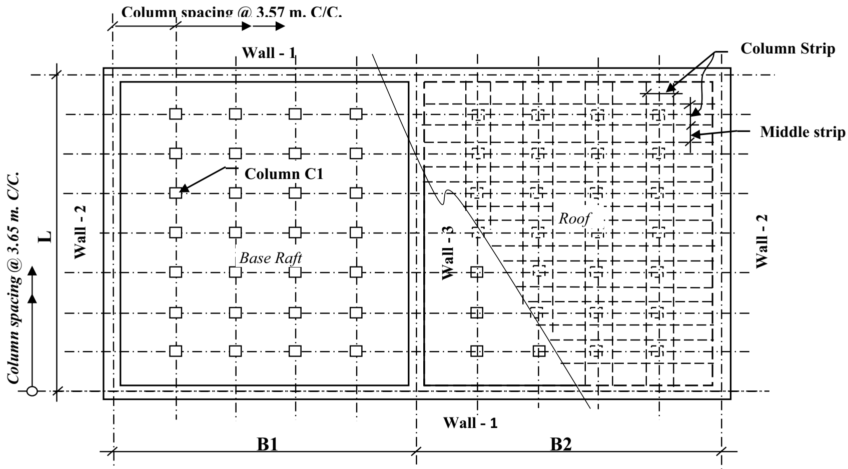


FIGURE 7.1 Plan view

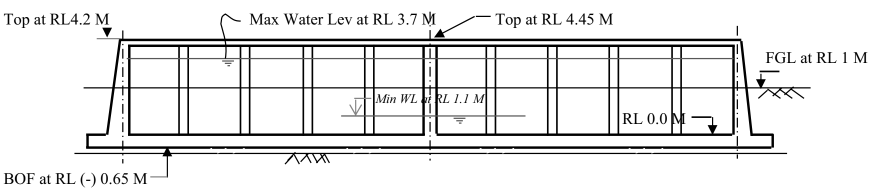


FIGURE 7.2 Sectional elevation

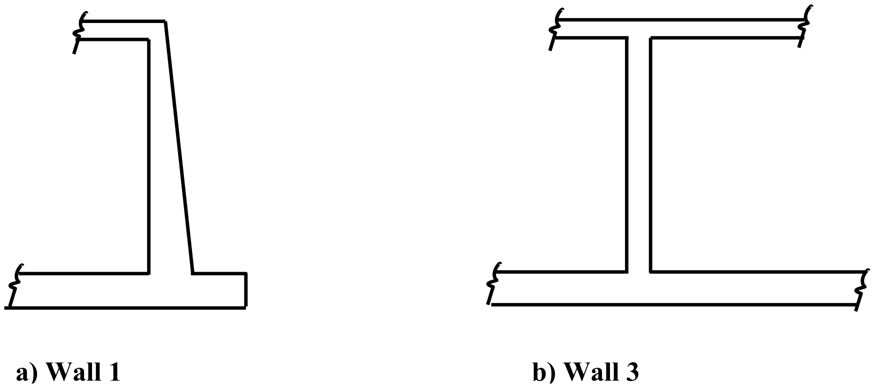


FIGURE 7.3 Wall sections

7.6 ROOF SLAB

The roof slab is designed as a flat slab resting over a peripheral wall and intermediate columns at equal spacing. There will be no drop slab at the top of the column head.

Input Data

Concrete Grade M25 Reinforcement bars fy = 415 MPa
Supports: RCC peripheral wall and Internal Columns without Drops.
Slab: Depth = 200 mm cover = 50 mm ϕdia = 12 mm deff = 144 mm

Column Head / Drop

Long direction = 300 mm Short direction = 300 mm

Wall thickness at top

Peripheral wall = 0.2 m Partition wall = 0.25 m

Slab dimensions

Along the x axis, Sx = 40.2 m (40 + 0.2)

Along the y axis, Sy = 25.325 m (25 + 0.25 × 0.5 + 0.2)

(c/c distance between supports.)

No. of panels (x -dir), npx = 11 nos

Number of panels (y -dir), npy = 7 nos

Longer span, L1 = 40.2 / 11 = 3.7 m

Shorter span, L2 = 25.325 / 7 = 3.6 m

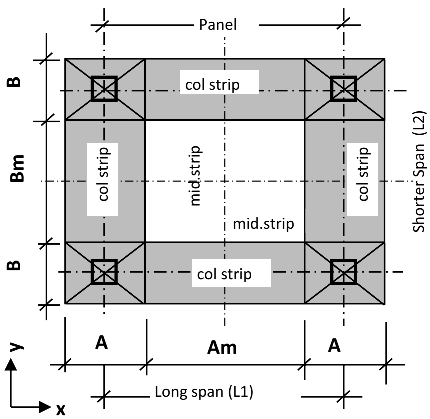


FIGURE 7.4 Flat slab panel

Column strip:

A = 1.8 m (2 × 0.25 × 3.6)
B = 1.8 m

Middle strip:

Am = 1.90 m (3.7–1.8)
Bm = 1.80 m (3.6–1.8)

Design moment for a span, M_0 = W. Ln / 8, where Ln = clear span extending from face to face of the column, but not less than 0.65L1. L1 is the length of span in the direction of moment, Mo.

Here, clear span (face to face of the column) = 3.7–0.3 = 3.4 m
And 0.65 L1 = 0.65 × 3.7 = 2.41 m
Ln = 3.40 m

Thickness of end wall supporting the roof slab, Dwall = 300 mm (equiv. thickness)
Height of end wall/internal columns, H wall = 4 m

Loads:

Dead Load		UDL_{DL}
Slab:	(200 / 1000) × 25 =	5.00 kN/m^2
Screed:	(50 / 1000) × 20 =	1.00
Waterproof Coating		0.12
		Total = 6.12 kN/m^2

Dead load UDL_{DL} = 6.12 kN/m^2
Live load UDL_{LL} = 1.5 kN/m^2

Limitations

The slab system will be designed by the direct design method subject to fulfilment of the following conditions:

i) *There shall be a minimum of three continuous spans in each direction*
In this case, the numbers of continuous panels are 11 (npx) and 7 (npy). Hence, okay.

ii) *The panels shall be rectangular, and the ratio of the longer span to the shorter span within a panel shall not be greater than 2.*
We have L1 / L2 = 3.7 / 3.6 = 1.03 < 2; okay.

iii) *The design live load shall not exceed three times the design dead load*
In this case, live load / dead load = 1.5 / 6.12 = 0.2 < 3; hence, okay.

Slab thickness

The thickness of the slab is checked by the determination of its shear capacity.
Total shear around the column head, Vs = 3.7 × 3.6 × (6.12 + 1.5) = 101 kN
deff = 144 mm. Column head dimensions = 300 mm × 300 mm
Load factor = 1.5 Limit state

Shear area, Ash = $2 \times [(300 + 144) + (300 + 144)] \times 144 / 10^6 = 0.26\ m^2$
Shear stress developed = $1.5 \times 101 / 0.26 = 583\ kN/m^2 = 0.58\ N/mm^2 < 1.25$; Safe.
Allowable shear stress = $0.25 \sqrt{fck} = 0.25 \sqrt{25} = 1.25\ N/mm^2$
So, Thickness of slab is within permissible limit.

Moment in Flat Slab

The flexural design of the slab will be done by the direct design method and distribution of moments between the column strip and middle strip in panels. Steps of design are as follows:

Step I: To determine total design moment for a span

Step II: To find out negative and positive design moments in end span and interior span.

Step III: Distribution of bending moments across the panel width

Step I: Total design moment for a span (unfactored) (IS 456: 2000–31.4.2.1)

Total design moment for a span shall be determined for a strip bounded laterally by the center line of the panel on each side of the center line of the supports.
Mslab, $\mathbf{M_0 = W_0 \times Ln / 8}$ where $W_0 = (UDL_{DL} + UDL_{LL}) \times L_2 \times Ln$
Total load, $W_0 = (6.12 + 1.5) \times 3.6 \times 3.4 = 93.27$ kN
Total moment, $M_0 = 93.27 \times 3.4 / 8 = 39.64$ kNm.

Step II: Negative and positive design moments (IS 456: 2000–31.4.3)

Design moment value = M_0 x Coefficient

End span

In the end span, the total design moment M_0 shall be distributed in the following proportion:

Interior negative design moment coefficient, A = $0.75-[0.10 / (1 + 1 / \alpha c)]$
Interior positive design moment coefficient, B = $0.63-[0.28 / (1 + 1 / \alpha c)]$
Exterior negative design moment coefficient, C = $0.65 / (1 + 1 / \alpha c)$

where αc is the ratio of flexural stiffness of the exterior columns/wall to the flexural stiffness of the slab at a joint taken in the direction moments is being determined.

$\alpha c = \Sigma Kc / Ks = 0.002 / 0.001 = 2.00$

$Kc = Iwall / Hwall = [3.6 \times (300 /1000)^3) / 12] / 4 = 0.002\ m^3$
$Ks = Islab / Lslab = [3.6 \times (200 / 1000)^3) / 12] /3.7 = 0.001\ m^3$
$A = 0.75-[(0.1 / (1 + 1 / 2)] = 0.68$
$B = 0.63-[(0.28 / (1 + 1 / 2)] = 0.44$
$C = 0.65 / (1 + 1 / 2) = 0.43$

Interior spans

In an interior span, the total design moment M_0 shall be distributed in the following proportion:

a) Negative design moment = 0.65 M_0
b) Positive design moment = 0.35 M_0

Step III: Distribution of bending moments between the column strips and middle strips across the panel width

Column Strip

a) Negative moment at an interior support = 75 % of the total negative moment in the panel at that support
b) Negative moment at an exterior support = 100 % of the total negative moment in the panel at that support
c) Positive moment for each span = 60 % of the total negative moment in the panel at that support.

Middle Strip

The middle strip shall be designed on the following bases:

a) The portion of the design moment not resisted by the column strip shall be assigned to the adjacent middle strip.
b) Each middle strip shall be proportioned to resist the sum of the moments assigned to its two half middle strips.
c) The middle strip adjacent and parallel to an edge supported by a wall shall be proportioned to resist twice the moment assigned to half the middle strip corresponding to the first row of interior columns.

TABLE 7.1
Bending moment coefficients

Moment coefficients in Flat Slab as per Cl no 31.4.3 IS 456 : 2000						**Drops Provided**			**NO**
		Value of coefficient							
		Interior panels		**Exterior panels**					
				Column supports			**Wall Supports**		
		Negative Bending Moment	**Positive Bending Moment**	**Negative Bending Moment**		**Positive Bending Moment**	**Negative Bending Moment**		**Positive Bending Moment**
				outer support	**inner support**		**outer support**	**inner support**	
Without drop	Column strip	0.49 *(0.65x0.75)*	0.21 *(0.35x0.6)*				0.22 *(0.43/3.6) x 1.8*	0.51 *0.68 x 0.75*	0.26 *0.44 x 0.6*
	Middle strip	0.16 (0.65x0.25)	0.14 (0.35x0.4)				0.23 (0.43 / 3.6) x 1.9	0.17 0.25 x 0.68	0.18 0.44 x 0.4

TABLE 7.2
Bending moment values (= M_0 x Coefficients) kNm:

Mslab, $M_0 = W_0 .Ln / 8$ W_0 93.27 KN Ln= 3.40m		Interior panels		Exterior panels					
				Column supports			Wall Supports		
		Negative Bending Moment	Positive Bending Moment	Negative Bending Moment		Positive bending moment	Negative Bending Moment		Positive bending moment
				outer edge	inner edge		outer edge	inner edge	
Without drops	column strip	19.32	8.32				8.52	20.22	10.46
	middle strip	6.44	5.55				9.00	6.74	6.98

Column strip bending moment diagram

Column Strip Width = 1.8 m

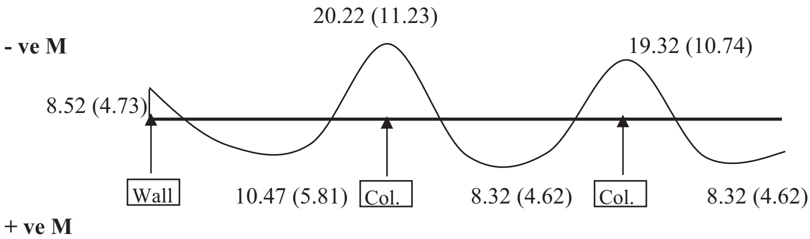


FIGURE 7.5(A) Bending moment diagram across column strip

Note: Values within () are moment in kNm per metre width.

Middle strip:

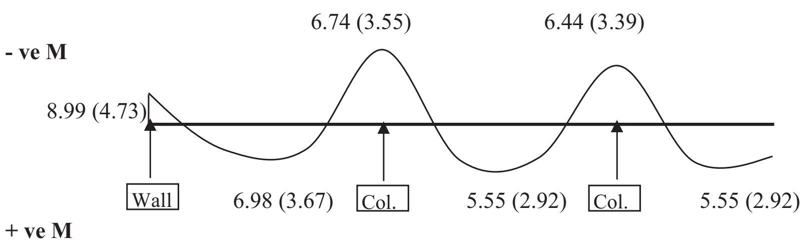


FIGURE 7.5(B) Bending moment diagram across middle strip

Note: Values within () are moment in kNm per metre width.

Reinforcement details are provided in construction drawing. Strength design done following the method of design of slab in Chapter 5.8.

7.7 SIDE WALLS

Input data

Top of Stem wall	TOP = 4.2 M	EL
Finished Grade level	FGL1 = 0.2 M	EL
Finished Grade level	FGL2 = 1 M	EL
Groundwater level	GWL = 0.5 M	EL
Bot of foundation	BOF = (-) 0.65 M	EL

Highest water level: HWL = 3.7 M EL *Basin level*: Basin = 0.0 M EL

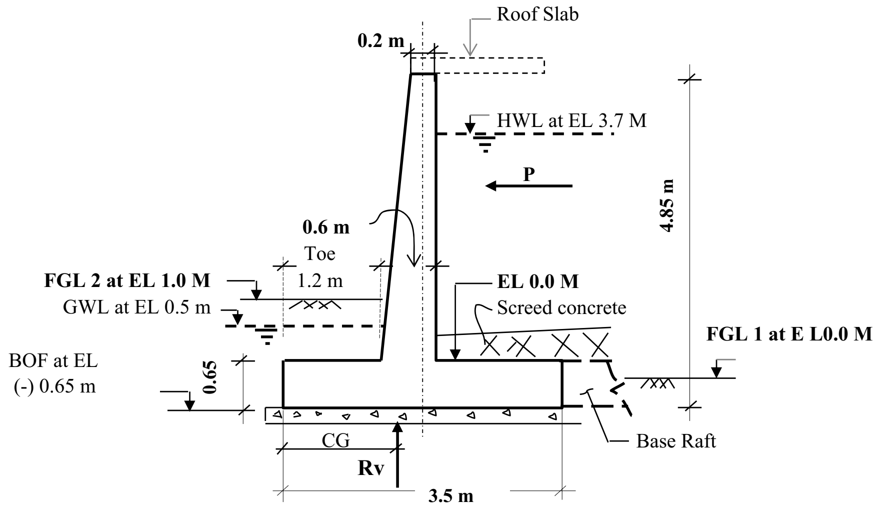


FIGURE 7.6 Wall Elevation

Ground condition and soil parameters:

Gross bearing pressure at the founding level, $p_{gross} = 340$ kN/m^2

Weight of backfill earth, $\gamma_{fill} = 18$ kN/m^3

Angle of internal friction of soil, $\phi_{fill} = 33$ degree Angle of cohesion, c = 0 kg/cm^2

Materials:

Concrete grade – M25 $\gamma_{conc} = 25$ kN/m^3 Reinf. Bars – fy = 415 MPa

Cover = 50 mm Max. bar diameter $\phi_{dia} = 20$ mm

Loading:

Ground surcharge $\gamma_{surch1} = 3.125$ kN/m^2 (~ 125 thick screed)

$\gamma_{surch2} = 10$ kN/m^2

Dimensioning:

Height	H = 4.85 m	Base width	B = 3.5 m
Base thickness	$B_T = 0.65$ m	Toe projection	Toe = 1.2 m (0.33 B)
Stem thickness at top	$S_{stop} = 0.2$ m		
Stem thickness at bottom	$S_{bot} = 0.6$ m	Heel = 1.7 m	

Method of RCC design:

Modular ratio method; m = 280 / (3 × 8.5) = 11

Earth pressure and Stability check.

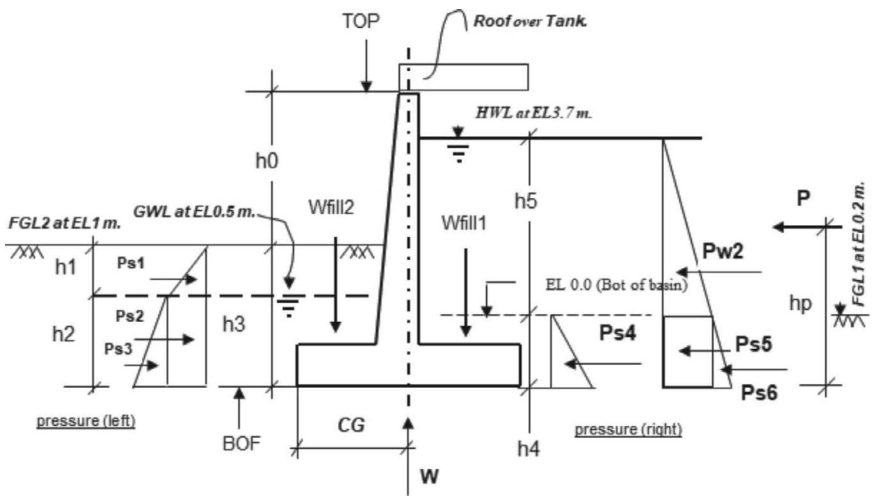


FIGURE 7.7 Loading on the wall

Stem ht. above FGL2	$h_0 = 3.2$ m	(4.2−1)
Height of fill above GWL	$h_1 = 0.5$ m	(1−0.5)
Submerged depth of fill	$h_2 = 1.15$ m	[0.5−(−0.65)]
	$h_3 = 1.65$ m	[1−(−0.65)]
	$h_4 = 0.65$ m	[0−(−0.65)]
	$h_5 = 3.7$ m	(3.7−0)

Coefficient of earth pressure:

Active, ka = 0.29 (1 − Sin ϕ) / (1 + Sin ϕ)

Passive, kp = 3.39

At rest, k_0 = 0.4

Lateral pressures acting on the wall:

	ID	Value	Unit	
⟶	Ps1	0.65	kN	$(0.5 \times 18 \times 0.5^2 \times 0.29)$
⟶	Ps2	6.34	kN	$[(18 \times 0.5 + 10) \times 0.29 \times 1.15]$
⟶	Ps3	1.53	kN	$[0.5 \times (18 - 10) \times 1.15^2 \times 0.29]$
⟶	Pw	6.61	kN	$(0.5 \times 10 \times 1.15^2)$
⟵	Pw2	68.45	kN	$(0.5 \times 10 \times 3.7^2)$
⟵	Ps4	0.49	kN	$(0.5 \times (18 - 10) \times 0.65^2 \times 0.29)$
⟵	Ps5	7.56	kN	$((10 \times 3.7 + 3.125) \times 0.29 \times 0.65)$
⟵	Ps6	2.11	kN	$0.5 \times 10 \times 0.65^2$)
	P	63.48	kN	*(Right to Left)*
	hp	1.96	m	

hp = [− Ps1 × (h2 + h1 / 3) − Ps2 × h2 / 2 − Ps3 × h2 / 3 − Pw × h2 / 3 + Pw2 × (h5/ 3 + h4) + Ps4 × h4 / 3 + Ps5 × h4 / 2 + Ps6 × h4/ 3] / P

Weight of backfilling soil + surcharge:
Wfill1 = [1.7 × (10 × 3.7 + 3.125 + (18 − 10) × (0.65 − 0.65) + 10 × (0.65 − 0.65))]
= 68.21 kN
Wfill2 = [1.2 × (10 + 18 × 0.5 + (18 − 10) × (1.15 − 0.65) + 10 × (1.15 − 0.65))]
= 33.60 kN

Weight of the RCC wall:

Stem Wstem = [(4.85 − 0.65) × 0.5 × (0.2 + 0.6)] × 25 = 42 kN
Base raft Wbase = (3.5 × 0.65 × 25) = 57 kN
Total gravity load: W = W stem + Wbase + Wfill1 + Wfill2 = 201 kN (= Rv)
Submerged unit wt. of soil, γ_{sub} = 8 kN/m^2

Load Case 1: Max water level inside reservoir + backfill outside

Reactions: Vertical upward, Rv = 201 kN
Lateral = Rh (=P) = 63.48 kN
CG location of resultant soil pressure (Rv) **= 1.19 m**

[Calculation of CG:

$PL = Ps1 + Ps2 + Ps3 + Pw$ =0.65 + 6.34 + 1.53 + 6.61 = 15.13 kN
$PR = Pw2 + Ps4 + Ps5 + Ps6$ =68.45 + 0.49 + 7.56 + 2.11 = 78.61 kN
$CG = (-PR \times hp + PL \times hp + Wfill2 \times 0.5 \times Toe + Wbase \times 0.5 \times B + Wstem \times (Toe + Sbot \times 0.5) + Wfill1 \times (Toe + Sbot + 0.5 \times Heel)) / (Wfill1 + Wfill2 + Wbase + Wstem)$

$Wfill2 \times 0.5 \times Toe = 33.6 \times 0.5 \times 1.2$	$= 20.2$ kN
$Wbase \times 0.5 \times B = 56.875 \times 0.5 \times 3.5$	$= 99.5$ kN
$Wstem \times (Toe + Sbot \times 0.5) = 42 \times (1.2 + 0.5 \times 0.6)$	$= 63$ kN
$Wfill1 \times (Toe + Sbot + 0.5 \times Heel) = 68.21 \times (1.2 + 0.6 + 0.5 \times 1.7)$	$= 181$ kN
$-PR \times hp = -78.61 \times 1.96$	$= -154$ kN
$+PL \times hp = +15.13 \times 1.96$	$= 29.7$ kN
	Sub Total = 239.03 kN

$Wfill1 + Wfill2 + Wbase + Wstem = 68.21 + 33.6 + 56.875 + 42 = 201$ kN
CG = 239.03 / 201 = 1.19 m.]

Stability check

a) Overturning:
Factor of safety (ovr) = Rv.CG/(Rh.hp) = 201 × 1.19 / (63.48 × 1.96) = 1.92 > 1.5
Minimum safety factor = 1.5 Hence; Safe.

b) Sliding (effect of passive resistance neglected):
Factor of safety (slide) = (W*μ + c_*100*B) / Rh
= (201 x 0.55 + 0 x 100 x3.5) / 63.48 = 1.74 > 1.5
Minimum safety factor = 1.5 Hence; Safe.
Coefficients of friction (suggested), μ = 0 55

TABLE 7.3
Summary of Stability Check.

Load case	Description of Load	W kN	P kN	hp m	CG m	μ	B m	FOS ovrturn	FOS slide
1	Inside full+ Backfill outside	201	63.48	1.96	1.19	0.55	3.5	2	11
2	Inside full + No Backfill outside	167	78.61	1.68	1.26	0.55	3.5	5	7
3	Inside Empty + Backfill outside	132	–15.1	0.50	1.44	0.55	3.5	6	>1.5

Note 1: P = (-) means left to right; FOS = Factor of safety

Design of RCC members.

TABLE 7.4
Calculated Gross bearing pressure:

Gross bearing pressure at founding level		Load case - 1: Inside full+ Backfill outside		Load case - 2: Inside full + No Backfill outside		Load case - 3: Inside Empty + Backfill outside	
Maxm. pressure	p_{max}	113	kN/m^2	87	kN/m^2	58	kN/m^2
Minm. pressure	p_{min}	2	kN/m^2	8	kN/m^2	18	kN/m^2
At Intermediate sections	p_1	75	kN/m^2	60	kN/m^2	44	kN/m^2
	p_2	56	kN/m^2	47	kN/m^2	37	kN/m^2

Note 1: Allowable gross bearing pressure = 340 kN/m^2
Note 2: Net upward pressure will be considered in the design of member section.

Calculation breakups:

$$pmax = (W / B) + ((6 \times W \times e) / (B^2))$$
$$= (201 / 3.5) + ((6 \times 201 \times 0.56) /(3.5^2))$$
$$= 113 \text{ kN/m}^2$$

where,e $= 0.5 \times B - CG$
$= 0.5 \times 3.5 - 1.19$
$= 0.56$m

$$pmin = (W / B) - ((6 \times W \times e) / (B^2))$$
$$= (201 / b) - ((6 \times 201 \times 0.56) / (3.5^2))$$
$$= 2.3 \text{ kN/m}^2$$

$$p1 = pmin + ((pmax - pmin) / B) \times (B - Toe)$$
$$= 2.3 + ((113 - 2.3) / 3.5) x (3.5 -1.2))$$
$$= 75 \text{ kN/m}^2$$

$$p2 = pmin + ((pmax - pmin) / B) \times (B - Toe - Sbot)$$
$$= 2.3 + ((113 - 2.3) / b) \times (3.5 - 1.2 - 0.6)$$
$$= 56.1 \text{ kN/m}^2$$

Design of RCC sections by modular ratio method

Permissible stresses in concrete:

As per IS: 456 working stress

Bending $\sigma_{bc} = 8.5$ N/mm^2
Compression $\sigma_c = 6.0$ N/mm^2
Tension $\sigma_t = 3.2$ N/mm^2

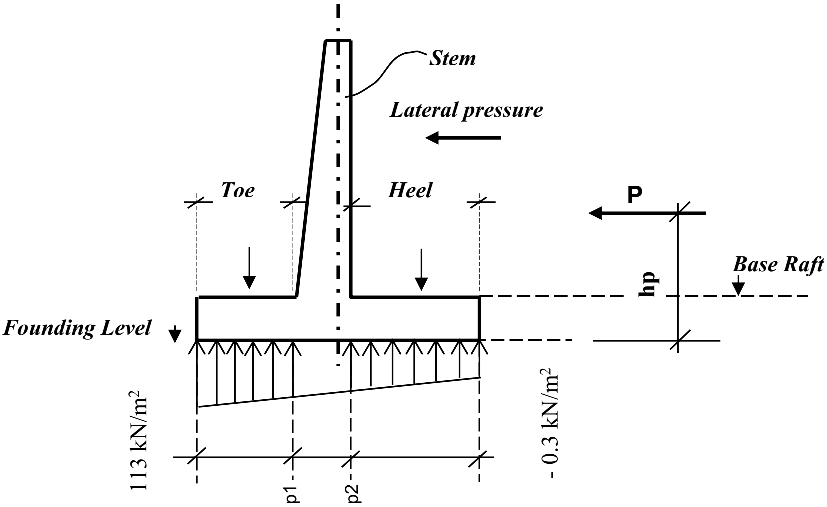


FIGURE 7.8 Base pressure diagram for load case 3.

As per Table 1 IS 3370 Part II) – resistance to cracking

Direct tension	$\sigma_{t3370} = 1.3$ N/mm^2
Bending tension	$\sigma_{bt3370} = 1.8$ N/mm^2
Shear	$\sigma_{sh3370} = 1.9$ N/mm^2

Permissible stresses in reinforcement (as per IS: 3370 Part II):

Direct tension:	$\sigma_{st} = 150$ N/mm^2	
Bending tension:	$\sigma_{bt} = 150$ N/mm^2 < 225 mm thick	
	$\sigma_{bt} = 190$ N/mm^2 > 225 mm thick	
Shear reinforcement:	150 N/mm^2 < 225 mm thick	
	190 N/mm^2 > 225 mm thick	
Modular ratio:	m = 11.00	
	k = 0.384	(m.σbc / m.σbc.σst)
	j = 0.87	(1 – k/3)
	R = 1.423	(0.5 σbc. k. j)

Unit:

Moment	kNm	σst	N/mm^2	Slab width, b =	1 m
Tension	kN	σbc	N/mm^2	Effective depth =	deff in

Base slab

Overall depth (Do) provided = 0.65 m

TABLE 7.5
Forces and moments

		Toe				Heel				
		Mtoe	Ftoe	T	C	Mheel	Fheel	T	C	
Load case	Description of load	kNm	kN	kN	kN	kNm	kN	kN	kN	Remarks
1	Inside full+saturated Backfill outside	55.70	47.19	0	0	–39.8	–46.8	0	0	(–) means tension at top
2	Inside full + no Backfill outside	41.45	35.12	0	0	–42	–49.4	0	0	
3	Inside empty + satu. Backfill outside	25.17	21.33	0	0	16.10	18.94	0	0	

Bending moment: Mtoe, Mheel   Axial tension = T
Shear: Ftoe, Fheel   Axial compression = C

Calculation backup

$$Mtoe = (0.5 \times (pmax + p\_1) - BT \times GammaConc) \times (Toe^2) / 2$$
$$= (0.5 \times (112.56 + 74.65) - 0.65 \times 25) \times (1.2^2) / 2$$
$$= 55.7kNm$$

$$Ftoe = (0.5 \times (pmax + p\_1) - BT \times GammaConc) \times (Toe - defft)$$
$$deff = (BT \times 1000 - cover - fdia \times 0.5) / 1000$$

$$deff = (0.65 \times 1000 - 50 - 20 \times 0.5) / 1000 = 0.59m$$
$$Ftoe = (0.5 \times (112.56 + 74.65) - 0.65 \times 25) \times (1.2 - 0.59)$$
$$= 47.2kN$$

$$MHeel = ((0.5 \times (pmin + p\_2) - BT \times GammaConc) \times (Heel^2) / 2) - (Wfill1 \times Heel / 2)$$
$$= ((0.5 \times (2 + 55.7) - 0.65 \times 25) \times (1.7^2) / 2) - (68.21 \times 1.7 / 2)$$
$$= -39.8 \ kNm$$

$$FHeel = ((0.5 \times (pmin + p\_2) - BT \times GammaConc) \times Heel) - Wfill1$$
$$((0.5 \times (2 + 55.7) - 0.65 \times 25) \times 1.7) - 68.21$$
$$-46.8kN$$

Reinforcement:

M_{design} = 55.70 kNm
Effective depth required, deff = $\sqrt{(M_{design}/R.b)}$ = 0.198 m
deff. provided = (0.65 × 1000 – 50 – 0.5 × 20) / 1000 = 0.59 m = 590 mm

xc = Do × 0.5 − (cover / 1000 + 0.5 × ϕdia / 1000)
= 0.65 × 0.5 − (50 / 1000 + 0.5 × 20 / 1000) = 0.27 m
j = 1 − k / 3 = 0.87

Ast = [{(MLQD − T × xc) × 10^6 / (σbt × j × deff)} + (T × 1000 / σst)] / 100
= [{(55.7 – 0 × 0.265) × 1000 / (190 × 0.87 × 0.59)} + (0 × 1000 / 150)] / 100
= 571 mm^2

Ast min = 0.0012 × (Do × 1000) × 1000 = 780 mm^2

Provide 16 mm diameter bars @ 200 mm c/c at top and bottom layers
Ast provided = 1006 mm^2 / m > Ast; Safe.

Moment of inertia of gross section including Reinf.(= m times area of steel):

Area of re-bars / face, Ast = 1006 mm^2
Asc = 1006 mm^2

Moment of inertia, MOI = 0.0240 mm^4

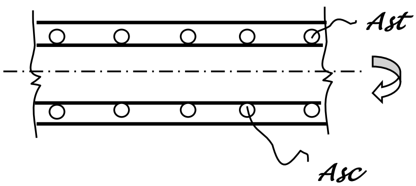


FIGURE 7.9 Re-bars in the slab

MOI = [(B × Do^3) / 12 + m × Asc × xc ^{2}] /10^{12}
= [(1000 × 650 ^{3}) / 12 +11 × 1006 × 267^2 + 11 × (1006 × 267 ^{2}] / 1012
= 0.024 m^4

b = 1000 mm D_0 = 650 mm   m = 11
xc = 0.5 × 650−50−0.5 × 16 = 267 mm
Calculated bending tension in concrete = σbt $_{conc}$
σbt $_{conc}$ = M_{design} (0.5 × Do / MOI) = 55.7 × (0.5 × 0.65 / 0.024) = 754 kN/m^2 = 0.754 N/mm^2
< 1.8 N/mm^2; permissible bending tension; Safe.

Stem

Overall depth at base = 0.6 m

TABLE 7.6
Forces and moments

		At base				At middle ht			
Load Case	Description of Load	Ms1 kNm	Fs1 kN	T kN	C kN	Ms2 kNm	Fs2 kN	T kN	C kN
1	Inside full + Saturated Backfill outside	108	54.97			Checked as propped cantilever; See Calculation below.			
2	Inside full + No Backfill outside	114	68.07						
3	Inside Empty + Backfill outside	-6.6	-13.1						

Bending moment: Ms1, Ms2 Axial tension: T
Shear: Fs1, Fs2 Axial compression: C
[Calculation backups: load case 1
Sum of moment at the base level = P × hp = 63.48 × 1.96 = 124.4 kNm
P = 63.48 kN Moment at top = 0 kNm
Height of stem = 4.2–(–0.65)–0.65 = 4.2 m
Total height above the base level = 4.2–(–0.65) = 4.85 m
Moment at base of stem = 124.42 × 4.2 / 4.85 = 107.7 kNm
Shear as base of stem = $63.48 \times 4.2^2 / 4.85^2 = 47.6$ kN
Shear as base of stem = 63.48 × 4.2 / 4.85 = 54.97 kN

Reinforcement at base: Tension inside

$M_{design} = 114$ kNm
Effective depth required, deff = $\sqrt{(M_{design}/R.b)} = 0.283$ m
deff. provided = 0.54 m = 540 mm
Ast. reqd = 1274 mm^2 Ast min = 648 mm^2
Provide 20 mm diameter bars @ 200 mm c/c at both faces
Ast provided = 1571 mm^2 / m > Ast. reqd; safe.

Moment of inertia of gross section including reinforcement (= m times area of steel):

Area of re-bars./ face, Ast = 1571 mm^2
Asc = 1571 mm^2

Moment of inertia, MOI = 0.020 mm^4

Refer to Figure 7.9 re-bars in the slab above,
$\sigma bt_{conc} = M_{design} (0.5 \times Do / MOI) = 114 \times (0.5 \times 0.60 / 0.020) = 1710$ kN/m^2 = 1.71 N/mm^2 < 1.8 N/mm^2; permissible bending tension; Safe.

Stem (check as propped cantilever)

a = 4.35 m L = 4.85 m
b = 0.5 m Rtop = 10.48 m
P = 63.48 kN x = 2.27 m

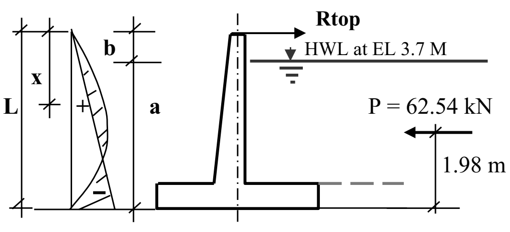


FIGURE 7.10 Bending moment diagram as a propped cantilever

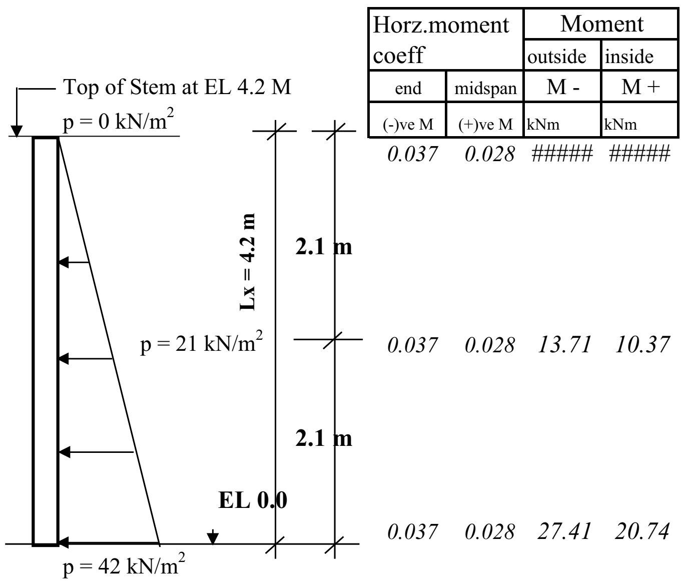
R top = $(63.48 \times 4.35^2) \times (5 \times 4.85 - 4.35) / (20 \times 4.85^3) = 10.48$ kN
x = $0.5 + (4.35^2 / 2 / 4.85) \times \sqrt{(1 - 4.35 / 5 / 4.85)} = 2.27$ m
Mspan = $10.48 \times 2.27 - (63.48 / 3 / 4.35^2) \times (2.27 - 0.5)^3 = 17.59$ kNm
Mbase = $(63.48 \times 4.35 / 60 / 4.85^2) \times (3 \times 4.35^2 - 15 \times 4.35 \times 4.85 + 20 \times 4.85^2)$
= 41.24 kNm.

M span = 17.59 kNm (positive moment)
Mbase = 41.24 kNm (negative moment)

Reinforcement at mid height: Tension outside

M_{design} = 17.59 kNm
Effective depth required, deff = $\sqrt{(M_{design}/R.b)}$ = 0.111 m
deff. Provided = 0.54 m
Ast reqd = 162 mm^2 Ast min = 648 mm^2
Provide 20 mm diameter bars @ 400 mm c/c at outer face
Ast provided = 786 mm^2 / m > Ast reqd; Safe.

Stem – horizontal re-bars



Lx = 4.2 m

FIGURE 7.11 Lateral forces and moments in the horizontal plane

Lx = 4.2 m Ly >25 m
Ly / Lx > 2 (long span moment coefficients are as per Table 26 of IS: 456)

At mid height

M1 = 0.037 × 21 × 4.2^2 = 13.71 kNm Tension outside
M2 = 0.028 × 21 × 4.2^2 = 10.37 kNm Tension inside

M_{design} = 13.71 kNm
Effective depth required, deff = $\sqrt{(M_{design}/R.b)}$ = 0.098 m

deff. Provided = 0.27 m
Ast reqd = 360 mm^2 Ast min = 324 mm^2
Provide 16 mm diameter bars @ 250 mm c/c at both faces.
Ast provided = 805 mm^2 / m > Ast reqd; safe.

At the bottom

M1 = 0.037 × 42 × 4.2^2 = 27.41 kNm Tension outside
M2 = 0.028 × 42 × 4.2^2 = 20.74 kNm Tension inside

M_{design} = 27.41 kNm
Effective depth required, deff = $\sqrt{(M_{design}/R.b)}$ = 0.139 m
deff. provided = 0.54 m
Ast reqd = 720 mm^2 Ast min = 648 mm^2
Provide 16 mm diameter bars @ 250 mm c/c at both faces.
Ast provided = 805 mm^2 / m > Ast reqd; Safe.

Wall 3: partition wall

The partition wall has been designed in the same way as done above considering water fill at one side.

7.8 BASE RAFT

FGL = RL 1 M Bottom of the foundation raft = RL (–) 0.65 M

Total weight of tank:

		Length	Breadth	Hh/ Th.	Area	Weight		
	Nos	m	m	m	sqm	kN		
Roof slab	1	40.4	50.4	0.2	2036	10181		
Wall 1	1	100	4	0.4	400	4000		
Wall 2	1	80	4	0.4	320	3200		
Wall 3	1	40	4	0.4	160	1600		
Columns	56	0.3	0.3	4		504		
Base raft	1	53.4	43.4		2318			
for walls 1 and 2	1	180.8	3.5	0.6	632.8	9492		
for wall 3	1	40.2	4	0.6	160.8	2412		
for internal columns and tank base				0.45	1524	17145		
						48533	kN	Subtotal (concrete)

A. RCCwork
B. Backfill earth around foundation – from EL (–) 0.65 M to FGL at EL 1.0 M.

		Length	Breadth	Ht/ Th.	Area	Weight		
	nos	m	m	m	sqm	kN		
Earthfill	1	184.8	1.2	1.65	222	6586		
(γ earth = 18 kN/m^3)						55119	kN	Subtotal (concrete + backfill)

C. Water fill up to max level – from RL 27 M to RL 30.7 M

Water	2	39.8	24.8	3.7	1974	73041		
						128160	kN	Subtotal (conc.+ backfill + water)

D. Ground surcharge around reservoir

Surcharge	1	184.8	1.2	222		2218		
(γ surch = 10 kN/m^2)					DL	130378	kN	Subtotal (conc.+b. fill+wtr+surch.)

E. Live load on roof (1.5 kN/m^2)

1	40.4	50.4	2036.2	LL	3054	kN

Pressure on the base raft:

Loading: tank full + fill around tank upto FGL + ground surcharge + LL on roof

Total downward force =	130378	kN	DL
	3054	kN	LL
Subtotal =	133432	kN	DL+LL
Base area =	2318	m^2	
Gross pressure on soil =	**57.57**	kN/m^2	< 340 kN/m^2; Safe.

Net design pressure on the base raft

a. Loading: tank full + fill around tank upto FGL + ground surcharge + LL on roof

Gross pressure on soil	57.574	
Less self-weight of the raft	−16.25 kN/m^2	(Slab thickness = 0.65m.)
Less water filling inside	−37	3.7 m height
Net design pressure =	**4.32** kN/m^2	

b. Loading: tank empty + no fill around tank + groundwater at FGL + LL on roof

Total downward force =	51587 kN	(48533 + 3054)
Base area =	2318 m^2	
Downward pressure on soil =	22.26 kN/m^2	
Groundwater pressure =	−16.5 kN/m^2	(ht. of water = 1.65 m.)
Gross pr. on soil = (Downward)	**5.76** kN/m^2	< 340 kN/m^2

Maximum local pressure on intermediate panels of base raft (during construction):

Downward pressure due to self-weight of the raft	= 13.25 kN/m^2 (slab thickness = 0.65m)
Groundwater pressure	= (−)16.5 kN/m^2 (height of water = 1.65m)
Net pressure	= (−) 0.25 kN/m^2 Uplift occurs.

Base raft will be designed for net pressure = 5.76 kN/m^2.

Load on intermediate columns

Load:	L	B	Influence area	UDL		Height	Axial	Moment (50% slab moment)
	m	m	sq.m	DL kN/m^2	LL kN/m^2	m	kN	kNm
Roof	3.66	3.62	13.22	6.12	1.5		101	
s/wt	0.3	0.3				4	9	
						P =	**110**	M = **5.35**

Check end wall with axial load + flat slab end moment per metre run

Load:	L	B	Influence Area	Thick	UDL		H	Axial	Moment
	m	m	sq.m	m	DL kN/m^2	LL kN/m^2	m	kN	kNm
at top	*(section 1000mm × 200mm)*								
Roof	1.83	1	1.83		6.12	1.5		14	
Self-weight	1			0.4			0.5	5	

						P =	*19*	M = *2.64*
							(Flat slab end moment)	
At the bottom	*(section 1000mm × 600mm)*							
Roof	1.83	1	1.83		6.12	1.5	14	
Self-weight	1			0.4		4	40	
						P =	*54*	M = *112*
							(Wall-1 base moment)	

Strength design of columns and walls have been done for above loads.

Design of base raft as flat slab

General design parameters

Refer to Figure 7.4 for plan view of the flat slab
Supports: RCC peripheral wall and internal columns without drops.

Slab dimensions:			
Along the x axis	Sx	40.2	m
Along the y axis	Sy	25.325	m
(c /c distance between supports)			
No. of panels (x direction)		11	nos
No. of panels (y direction)		7	nos
Longer span	L1	3.7	m
Shorter span	L2	3.6	m

Column strip:					
A	1.80	m	**Depth of slab:**	Dslab	650 mm
B	1.80	m		Cover	50 mm
Middle strip:				deff	600 mm
Am	1.90	m	**Column head/drop:**		
Bm	1.80	m	Column size- long		300mm
Ln	**3.40**	m	Column size- Short		300mm
Materials:			(no drop provided)		
Concrete grade	M25				
Unit weight of concrete	γconc	25 kN/m³			
	fck	25 MPa			
Reinforcement bars	fy	415 MPa			

Thickness of the end wall supporting the roof slab,	Dwall	= 300 mm (average thickness)
Height of end wall/internal columns,	Hwall	= 4 m

Design load:

Dead load	UDL_{DL}	4.26 kN/m²	(5.76−1.5)
Live load	UDL_{LL}	1.5 kN/m²	
Net design pressure =		= 5.76 kN/m²	

Method of design as per IS: 456–2000

i) No of continuous spans in each direction – minimum	7 > 3;	okay
ii) Ratio of longer span to shorter span – minimum	1.03 < 2;	okay
iii) Ratio of design live load to design dead load	0.4 < 3;	okay

So, the slab will be designed by the direct design method as per code.

Depth of slab

Shear stresses:

Effective shear area near column/drop

Total shear = 3.7 × 3.6 × 5.76 = 76.72 kN

Effective shear area near column base = 2 [(300 + 600) + (300 + 600)] × 600 =2160000 mm²

Shear stress, τv = 76720 / 2160000 = 0.04 N/mm²

Allowable shear stress in working stress, $\tau c = 0.16 \sqrt{fck} = 0.8$ N/mm² > τv; safe

Observations:

shear stress in the flat slab is within the permissible limit.

Bending moment (asper IS: 456–2000)

Mslab, $M_0 = W_0 \times Ln / 8$ where $W_0 = (UDL_{DL} + UDL_{LL}) \times L2 \times Ln$

Total load, $W_0 = (4.26 + 1.5) \times 3.6 \times 3.4$ = 70.50 kN

Total moment, $M_0 = 70.5 \times 3.4 / 8$ = 29.96 kNm

Design moment value = M_0 × Coefficient

Bending moment coefficients

End span

$\alpha c = \Sigma Kc / Ks = 0.09$

$Kc = [3.6 \times (300/1000)^3] / 12 / 4 = 0.002$ m³

$Ks = [3.6 \times (650/1000)^3] / 12 / 3.7 = 0.022$ m³

Interior negative design moment coefficient, A = 0.7417

Interior positive design moment coefficient, B = 0.6067

Exterior negative design moment coefficient, C = 0.0542

Interior spans:

a) – ve design moment =0.65 *b) + ve design moment =0.35*

TABLE 7.7
Bending Moment Coefficients.

As per Cl.no 31.4.3		Interior panels		Exterior panels: Column supports			Exterior panels: Wall Supports		
Drops Provided: NO; Value of coefficients		Negative bending moment	Positive bending moment	Negative bending moment: outer edge	Negative bending moment: inner edge	Positive bending moment	Negative bending moment: outer edge	Negative bending moment: inner edge	Positive bending moment
Without drops	column strip	0.49	0.21				0.027	0.556	0.364
	middle strip	0.16	0.14				0.029	0.185	0.243

TABLE 7.8
Bending Moment Values (= Mo × Coefficients) kNm :

Mslab, M0 = W0 . Ln / 8 W0 = 70.50 KN Ln= 3.40 m		Interior panels		Exterior panels: Column supports			Exterior panels: Wall Supports		
		Negative bending moment	Positive bending moment	Negative bending moment: outer edge	Negative bending moment: inner edge	Positive bending moment	Negative bending moment: outer edge	Negative bending moment: inner edge	Positive bending moment
Without drops	column strip	14.61	6.29				0.81	16.67	10.91
	middle strip	4.87	4.19				0.86	5.56	7.27

Design moment

Column strip bending moment diagram

Column strip Width = 1.8 m

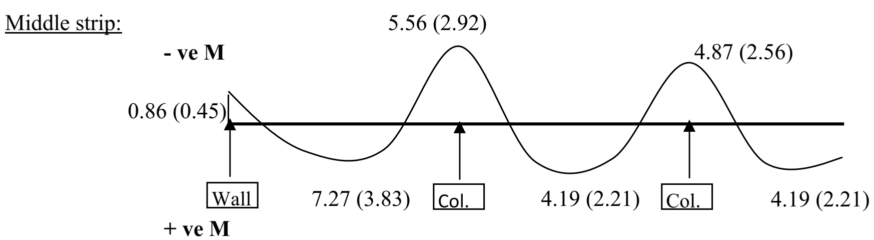


FIGURE 7.12(A) Bending moment diagram of column strip

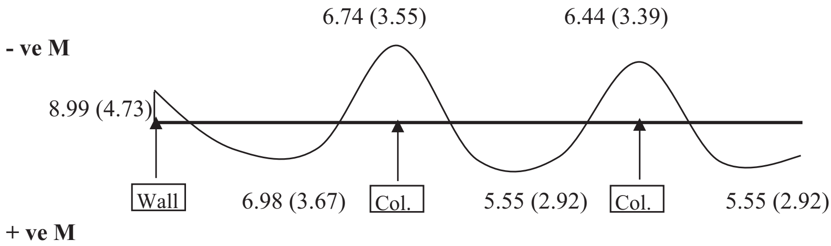


FIGURE 7.12(B) Bending moment diagram of the middle strip

Note: Values within () are moment in kNm per metre width.

Reinforcement details are provided in construction drawing. The strength design of the slab has been done following the method of design of the slab in chapter 5.8.

7.9 SETTLEMENT

The safe bearing pressure was recommended as 340 kN/m^2, for 25mm settlement in geotechnical report. In this case, the estimated gross pressure is 57.57 kN/m^2. Hence, the expected total settlement should not exceed 4.5mm ($25 \times 57.57 / 340 = 4.23$ mm), and differential settlement is less than 1 in 1000.

However, the designer should calculate actual settlement for the service period considering the engineering properties of the soil stratum below the reservoir.

7.10 CONSTRUCTION DRAWING

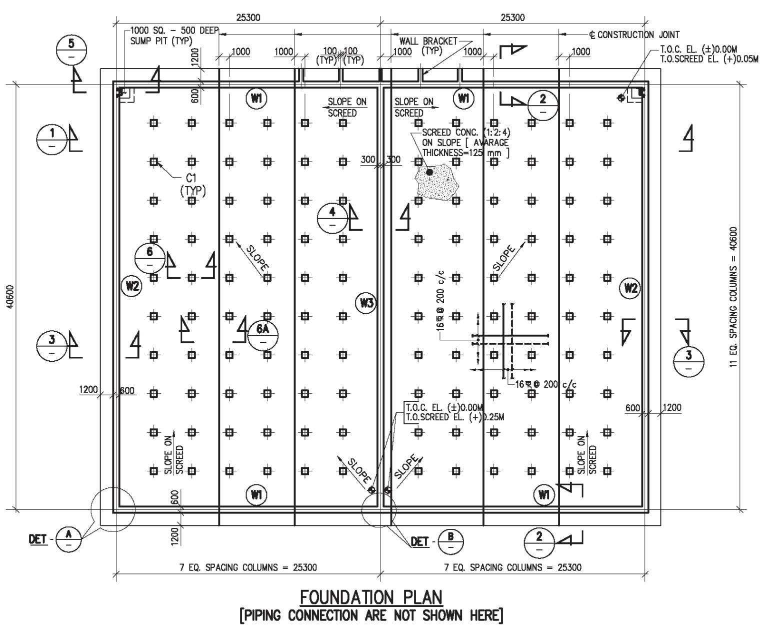


FIGURE 7.13(A) Sheet 1

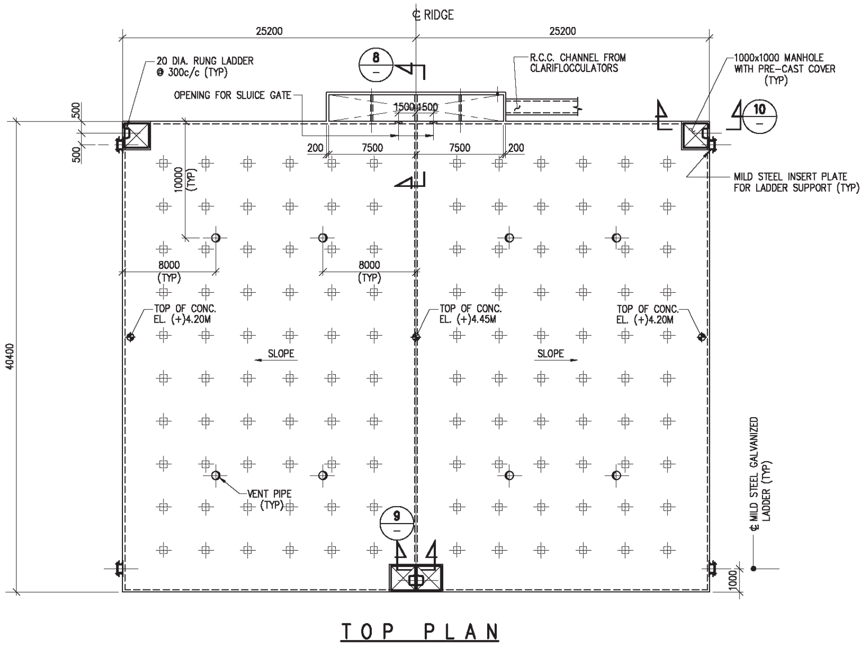


FIGURE 7.13(B) Sheet 2

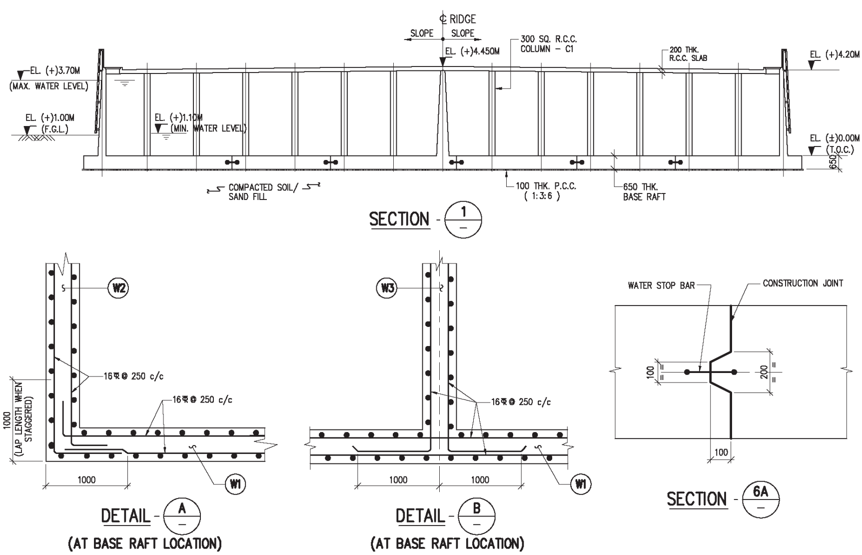


FIGURE 7.13(C) Sheet 3

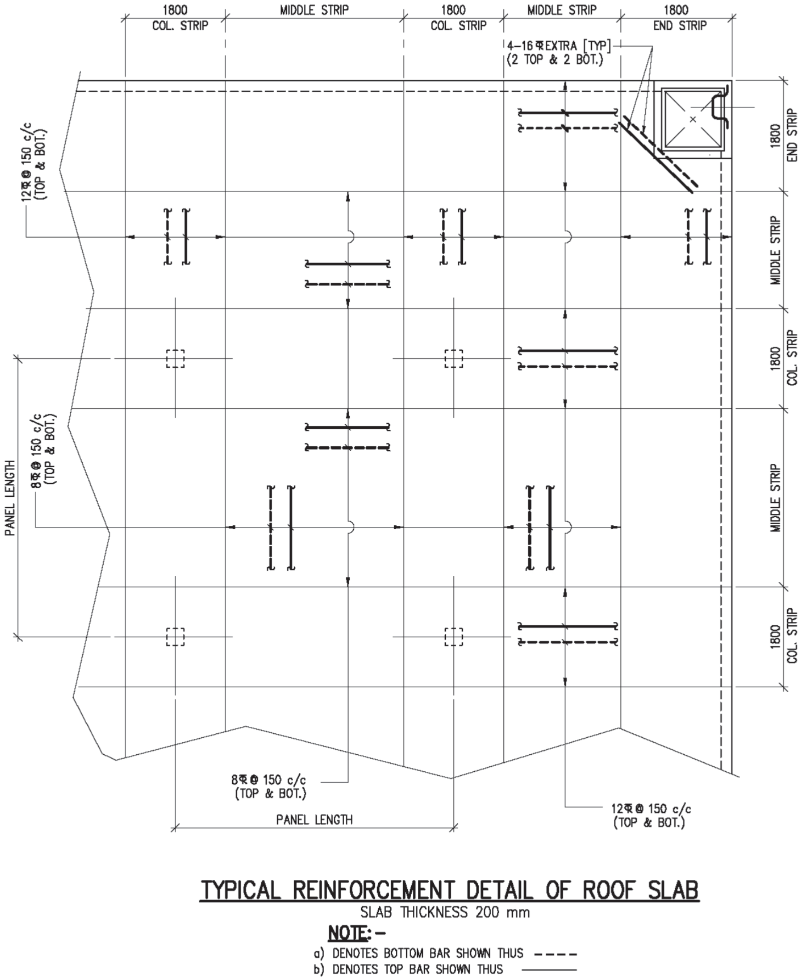


FIGURE 7.13(D) Sheet 4

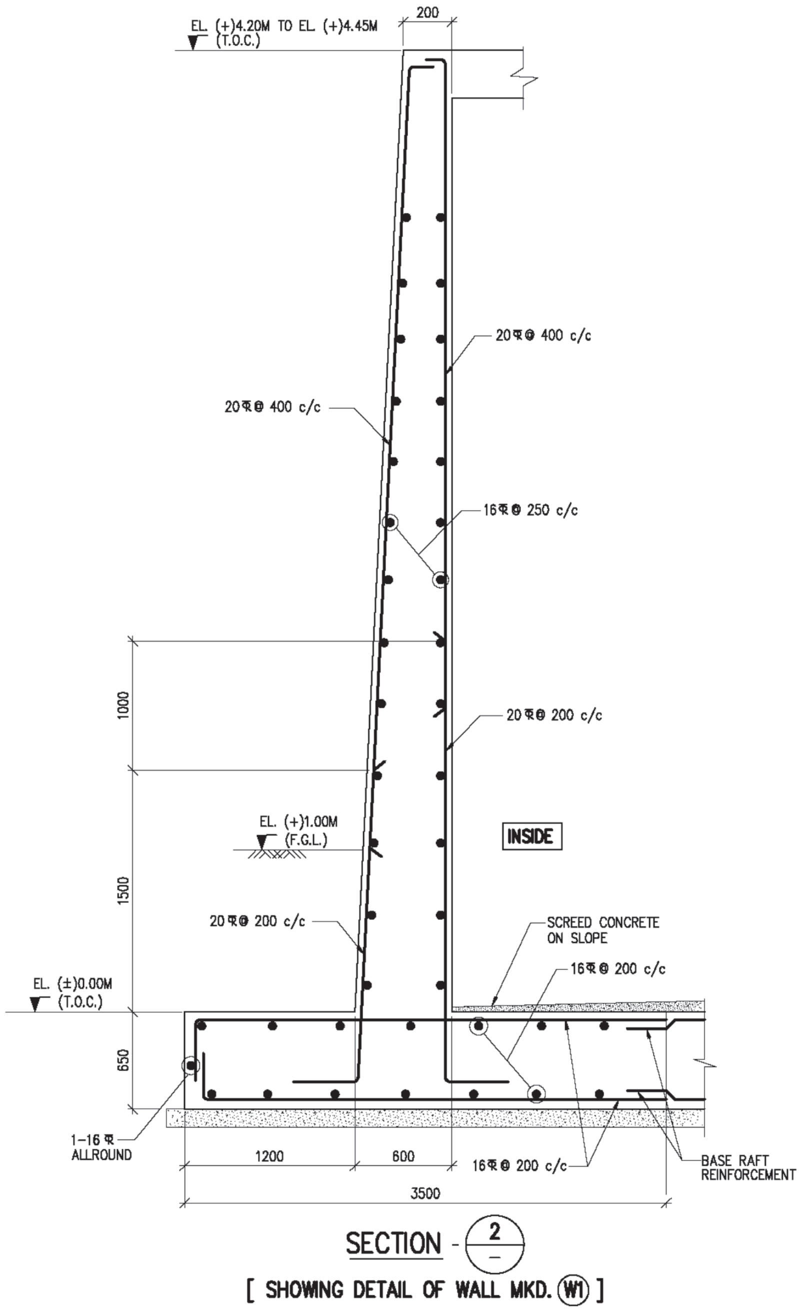


FIGURE 7.13(E) Sheet 5

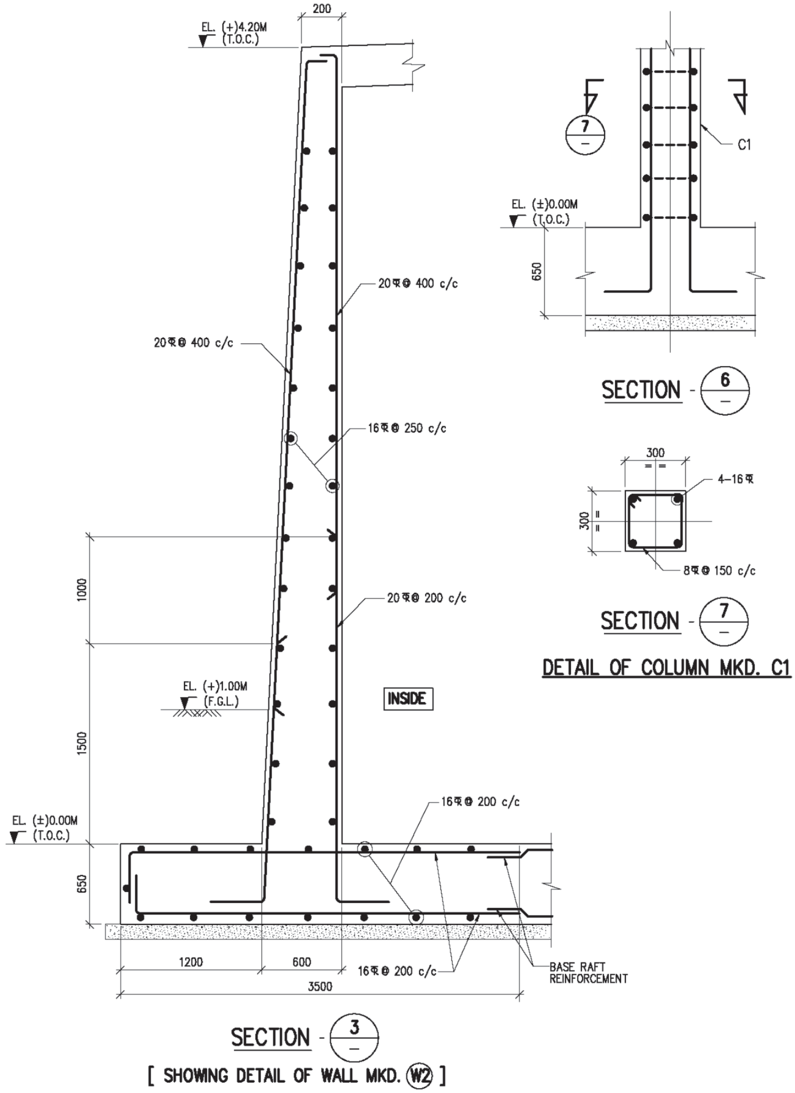


FIGURE 7.13(F) Sheet 6

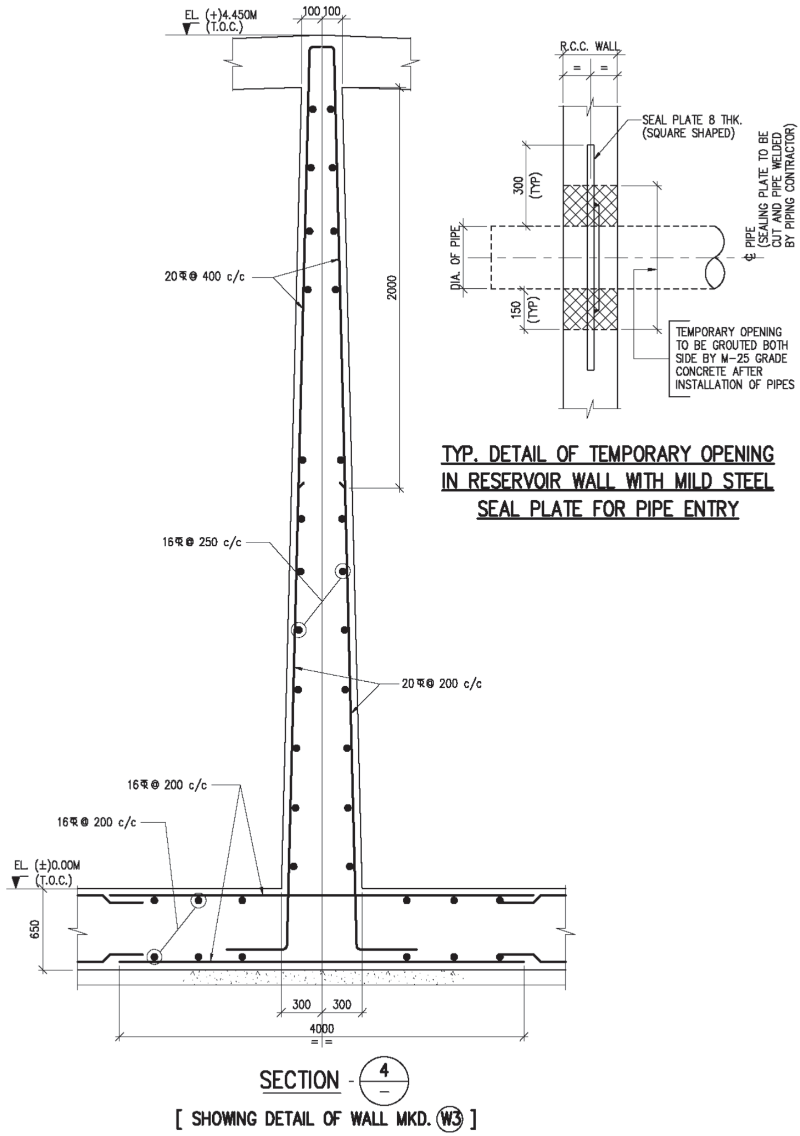


FIGURE 7.13(G) Sheet 7

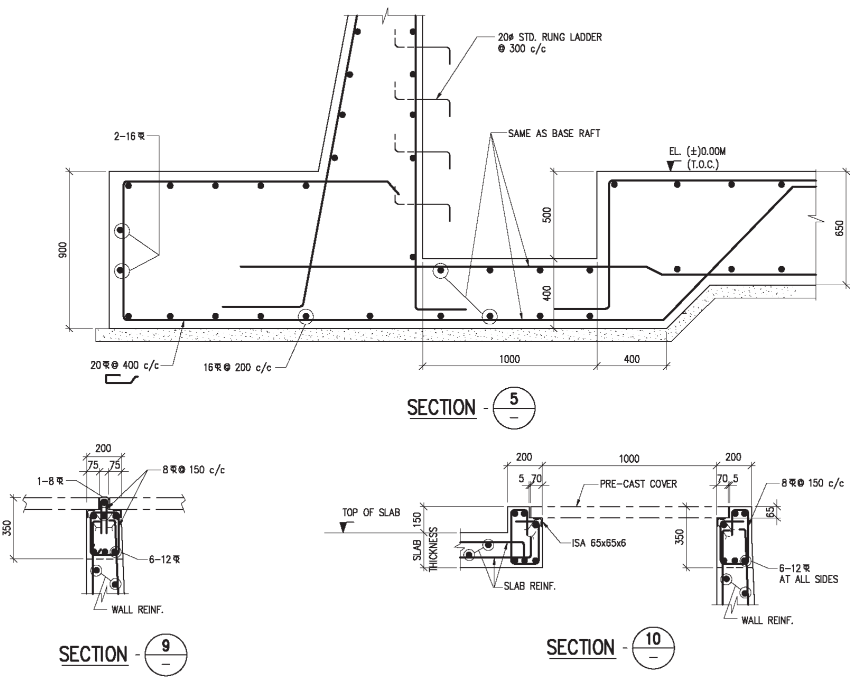


FIGURE 7.13(H) Sheet 8

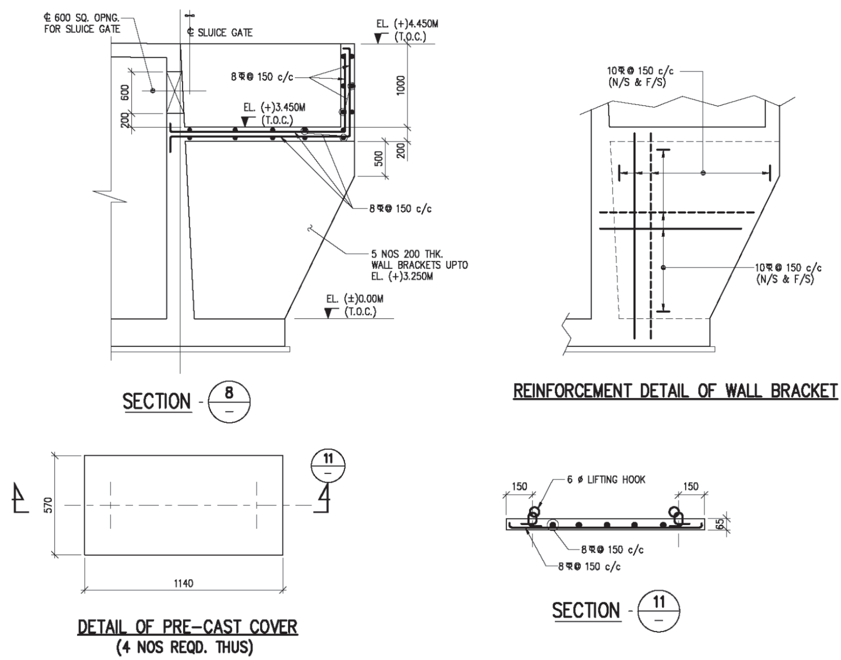


FIGURE 7.13(I) Sheet 9

NOTES :

1. ALL DIMENSIONS ARE IN MILLIMETRE AND ELEVATIONS ARE IN METER.
2. ALL CONCRETE WORK SHALL BE DONE IN ACCORDANCE WITH IS:456.
3. GRADE OF CONCRETE SHALL BE M25 (DESIGN MIX).
4. REINFORCEMENT BARS SHALL BE Fe 415 (YIELD STRESS, $Fy = 415\ N/mm^2$).
5. THE CLEAR COVER TO REINFORCEMENT SHALL BE AS UNDER:
   a) BASE RAFT : 50mm (TOP & SIDES), 75mm (BOTTOM)
   b) WALL : 40mm
   c) SLAB : 20mm
6. THE REINFORCEMENT AND MEMBER SIZES GIVEN HERE ARE PRELIMINARY AND NOT FOR CONSTRUCTION.

LEGEND :

1. F.G.L : FINISH GRADE LEVEL
2. T.O.C : TOP OF CONCRETE
3. P.C.C : PLAIN CEMENT CONCRETE
4. R.C.C : REINFORCEMENT CEMENT CONCRETE
5. BOT. : BOTTOM
6. THK. : THICKNESS
7. TYP : TYPICAL
8. c/c : CENTRE TO CENTRE
9. ℄ : CENTRE LINE
10. N/S : NEAR SIDE
11. F/S : FAR SIDE

FIGURE 7.13(J) Sheet 10

REFERENCES

Bureau of Indian Standards (BIS). *Concrete Structures for Storage of Liquids – IS 3370 (Part 1&2): 2009.* New Delhi: 9 Bahadur Shah Zafar Marg, Pin 110002, 1st Revision, 2009.

Bureau of Indian Standards (BIS). *Indian Standard Plain and Reinforced Concrete – IS 456:2000.* New Delhi: 9 Bahadur Shah Zafar Marg, Pin 110002, 4th Revision, 2000.

Indian Standard Institution. *Design Aids for Reinforced Concrete SP:16-1980.* New Delhi: 110002, 2th Revision, 1980.

8 Design of a two-story office building on soft soil

8.1 DESCRIPTION OF BUILDING

This document contains the analysis and design of a two-story reinforced concrete office building for a printing factory. The ground floor will be used for offices and facilities, while the first floor is designated for offices and storage. The building is well-ventilated and illuminated, with access stairs and an adequate number of doors for the easy movement of persons. The building framework has been designed to meet Dead, Live, and Imposed loads, as well as wind/seismic requirements according to Indian Standard Codes of Practice. All foundations will rest on soil.

8.2 REFERENCE CODES AND STANDARD

a) IS 456: 2000 Plain and reinforced cement concrete.
b) IS 875: 1987 Design loads for building and structures.
c) IS 1893: 2002 Criteria for earthquake-resistant design of structure.
d) SP 16 Design aids to IS: 456
e) Soil investigation report (site-specific)

8.3 MATERIAL USED

a) Grade of concrete: M25 (characteristic strength = 25 MPa cube strength at 28 days)
b) Reinforcement bars: Fe 415 as per IS: 1786 (yield stress = 415 MPa)

8.4 LOAD AND LOAD COMBINATIONS

Design parameters:

SBP =	61 kN/m^2	Ground floor =	0 M	Grade level =	−0.6 M
γsoil =	18 kN/m^2	First floor =	3.75 M		
γconc =	24 kN/m^2	Roof =	7.5 M		

DOI: 10.1201/9781003618119-8

Load intensity:

			DL	LL
Roof	125 mm	Slab	3 kN/m²	
	70 mm	Roof treatment	1.68	
	12 mm	Plaster	0.24	
		Miscellaneous	0.08	
		Sum =	5 kN/m²	1.5 kN/m²
		(*) self-weight of framing members =	2.72 kN/m²	
		Total intensity =	7.72 kN/m²	

			DL	LL
Floor	125 mm	Slab	3 kN/m²	
	50 mm	Floor finish	1.2	
	12 mm	Plaster	0.24	
		Partition wall	1	
			5.44 kN/m²	3 kN/m²
			2.72	
		Intensity =	8.16 kN/m²	

[*Note: (*) Self-weight of framing members are auto-generated in STAAD analysis. So, this load will not appear in floor load intensity given in STAAD input.]*

Roof over stair	**DL**	**LL**
	5.00 kN/m²	1.5 kN/m²

Stairs

Waist slab		**DL**	**LL**
178 mm	slab	4.27 kN/m²	
40 mm	Floor finish	0.96	
12 mm	Plaster	0.24	
Hung + partition wall		0	
		5.47 kN/m²	5 kN/m²

Landing slab		**DL**	**LL**
178 mm	Slab	4.27 kN/m²	
40 mm	Floor finish	0.96	
12 mm	Plaster	0.24	
Hung + partition load		0	
		5.47 kN/m²	5 kN/m²

Stair well 3.6 m × 5.7 m Floor height 5.5 m

	nos	L	B	UDL	DL
Waist slab	3	2.51	1.5	5.47	61.92
Landing slab	2	1.5	1.5	5.47	24.62
					86.54 kN.

Equiv. load in stair well: 86.54 / (3.6 x 5.7) = **4.22** kN/m^2 **DL** < floor DL

LL = **5.00** kN/m^2

Brick wall load:

	20% opng		Ht		Th		Unit wt
Parapet =	1	×	1	×	0.125	×	20
Ext. wall above first floor =	0.8	×	3.15	×	0.125	×	20
External wall above GF =	0.8	×	3.15	×	0.125	×	20

Water tank on roof = 100 kN.
Equivalent UDL 2.8 kN/m^2

Wind load

Wind intensity = 1.21 kN/m^2 Force coefficient = 1.2
Building height above EGL = 8.1 m
Building length = 14.9 m

Total wind force = 175 kN

Wind force is less than seismic, hence not considered for analysis; seismic load governs.

Determination of design wind pressure as per IS 875 (Part3): 2015

Wind speed, $Vz = Vb \times k1 \times k2 \times k3 \times k4 =$ 55.78 m/sec

Wind pressure, $pz = 0.6 \times Vz^2 = 1.87$ kN/m^2

Vb =	**50** m/s	Kd =	0.9
k1 =	1	Ka =	0.8
k2 =	0.97	Kc =	0.9
k3 =	1		
k4 =	1.15		

Design wind pressure, $pd = Kd \times Ka \times Kc \times pz =$ **1.21** kN/m^2

Seismic load

Seismic coefficient: as per IS 1893 Part 1: 2016

Depth frame girder = 0.6 m

Height = 7.2 m [CG of beams above FFL]

Ta =0.075 × $^{0.75}\sqrt{h}$ = 0.33

αh = Z. I. Sa / (2 R. g) = 0.9

Z =	0.36
I =	1
R =	5 SMRF
Sa /g =	2.5
I/R =	0.2

Roof / floor dimension:
Length = **11** m. Width= **14.9** m. Total area = 164 sq.m

Seismic weight calculation for frame design.

Roof: Intensity = 7.72 kN/m^2 Area = 164 sqm.

	nos	L(m)	W(m)	H(m)	% opng		DL(kN)	LL(kN)	Sum(kN)
Roof slab including framing members						7.72 × 164=	1265	0	
Parapet	1	51.8	0.125	1			129.5		
Outer wall	1	18.6	0.125	1.575	0.8		58.6		
Water tank							5		
Columns	12	0.5	0.3	1.575			71		
							264	0	264

No of nodes = 12

Nodal load = 264 × 0.09 / 12 = 2 kN

Floor: Intensity, DL = 8.16 kN/m^2 Area = 164 sqm.
LL = 0.75 kN/m^2 [25% considered for Seismic mass calculation as per IS 1893]

	nos	L(m)	W(m)	ht(m)	% opng	DL(kN)	LL(kN)	Sum(kN)
Floor			8.16 × 164 =			1337	122.9	
Outer wall	1	51.8	0.13	4.95	0.8	512		
Columns	12	0.5	0.3	3.45		155		
						2005	123	2128

No of nodes = 12 Nodal load = 2128 × 0.09 / 12 = 16 kN

Seismic load distribution two-story office

		DL	LL	hi	Wi	Wi.hi²	Qi	No of Nodes
W:	Roof	264	0	9.89	264	25819	52.41	16
	First floor	2005	123	6.14	2128	80224	163	12
		2269 +	123			106043		
		=	2392 kN					

$Qi = Vb \times Wi\, hi^2 / \Sigma\, Wi\, hi^2$

$Vb = \alpha h \times W = (2269 + 123) \times 0.09 = 215$ kN

Applied load at roof nodes = 52.41 / 16 = 3.28 kN (including stair roof)
Applied load at floor nodes = 163 / 12 = 13.57 kN

Load combinations

1 DL	Dead weight of structures
2 LL	Live load
3 SLZ	Seismic load along the transverse direction
4 SLX	Seismic load along the longitudinal direction

5 DL + LL
6 DL + SLZ
7 DL − SLZ
8 DL + SLX
9 DL − SLX
10 0.75 (DL + 0.5LL+SLZ)
11 0.75 (DL + 0.5LL−SLZ)
12 0.75 (DL + 0.5LL+SLX)
13 0.75 (DL + 0.5LL−SLX)

Working stress (unfactored load)

14 1.5 DL + 1.5 LL
15 1.5 DL + 1.5 SLZ
16 1.5 DL − 1.5 SLZ
17 1.5 DL + 1.5 SLX
18 1.5 DL − 1.5 SLX
19 1.2 (DL + LL + SLZ)
20 1.2 (DL + LL − SLZ)
21 1.2 (DL + LL + SLX)
22 1.2 (DL + LL − SLX)

Limit state (factored)

8.5 STRUCTURAL ANALYSIS AND MEMBER DESIGN

Structural analysis has been done using STAAD PRO software. Input data are given below for the reader's understanding.

8.5.1 Analytical model (STAAD)

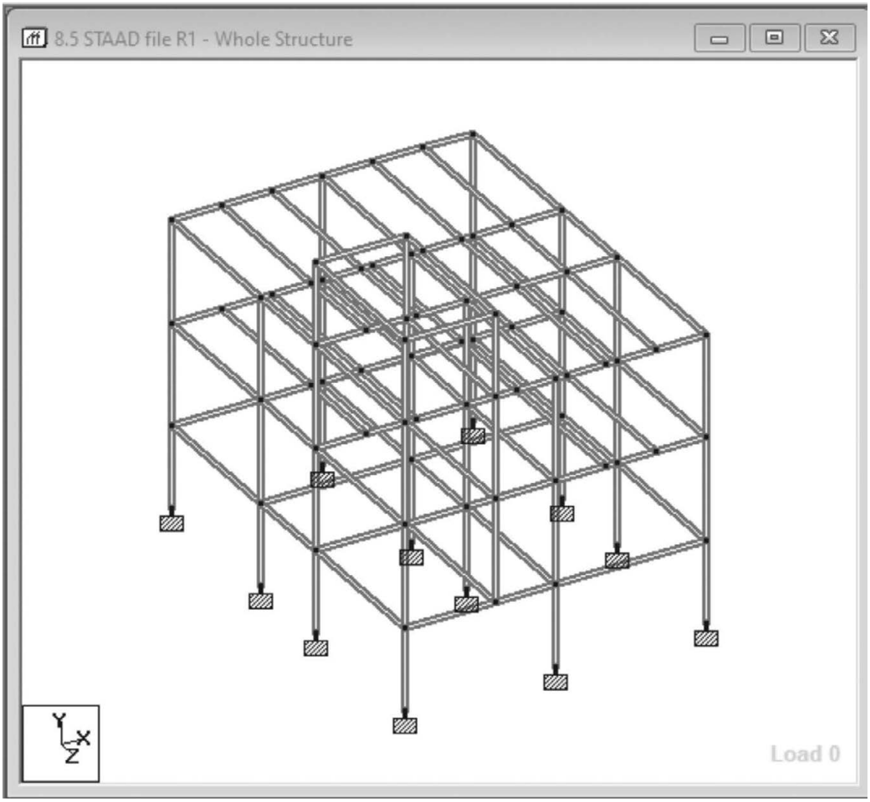


FIGURE 8.1 Analytic model of building framework

8.5.2 STAAD input command file

```
STAAD SPACE OFFICE BLDG
START JOB INFORMATION
ENGINEER DATE 04-Nov-16
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN

JOINT COORDINATES
5 22 0 0.47; 6 27.5 0 0.47; 7 33 0 0.47; 13 22 2.99667 0.47;
14 27.5 2.99667 0.47; 15 33 2.99667 0.47; 22 27.5 6.74333 0.47;
```

23 33 6.74333 0.47; 37 22 0 6.07167; 38 27.5 0 6.07167; 39 33 0 6.07167;
45 22 2.99667 6.07167; 46 27.5 2.99667 6.07167; 47 33 2.99667 6.07167;
54 27.5 6.74333 6.07167; 55 33 6.74333 6.07167; 69 22 0 9.55833;
70 27.5 0 9.55833; 71 33 0 9.55833; 77 22 2.99667 9.55833;
78 27.5 2.99667 9.55833; 79 33 2.99667 9.55833; 86 27.5 6.74333 9.55833;
87 33 6.74333 9.55833; 101 22 0 15.16; 102 27.5 0 15.16; 103 33 0 15.16;
109 22 2.99667 15.16; 110 27.5 2.99667 15.16; 111 33 2.99667 15.16;
118 27.5 6.74333 15.16; 119 33 6.74333 15.16; 153 23.8333 6.74333 0.47;
154 23.8333 6.74333 6.07167; 155 23.8333 6.74333 9.55833;
157 25.6667 6.74333 0.47; 158 25.6667 6.74333 6.07167;
159 25.6667 6.74333 9.55833; 161 29.3333 6.74333 0.47;
162 29.3333 6.74333 6.07167; 163 29.3333 6.74333 9.55833;
164 29.3333 6.74333 15.16; 165 31.1667 6.74333 0.47;
166 31.1667 6.74333 6.07167; 167 31.1667 6.74333 9.55833;
168 31.1667 6.74333 15.16; 357 22 6.74333 15.16; 358 27.5 10.4933 0.47;
359 33 10.4933 0.47; 360 27.5 10.4933 6.07167; 361 33 10.4933 6.07167;
362 27.5 10.4933 9.55833; 363 33 10.4933 9.55833; 364 27.5 10.4933 15.16;
365 33 10.4933 15.16; 366 23.8333 10.4933 0.47; 367 23.8333 10.4933 6.07167;
368 23.8333 10.4933 9.55833; 370 25.6667 10.4933 0.47;
371 25.6667 10.4933 6.07167; 372 25.6667 10.4933 9.55833;
374 29.3333 10.4933 0.47; 375 29.3333 10.4933 6.07167;
376 29.3333 10.4933 9.55833; 377 29.3333 10.4933 15.16;
378 31.1667 10.4933 0.47; 379 31.1667 10.4933 6.07167;
380 31.1667 10.4933 9.55833; 381 31.1667 10.4933 15.16; 382 22 10.4933 15.16;
385 22 10.4933 9.55833; 386 22 10.4933 6.07167; 387 22 10.4933 0.47;
388 22 6.74333 9.55833; 389 22 6.74333 6.07167; 390 22 6.74333 0.47;
391 25.2967 6.74333 9.55833; 392 25.2967 6.74333 15.16;
393 25.2967 10.4933 9.55833; 394 25.2967 10.4933 15.16;
399 25.2967 2.99667 15.16; 402 25.2967 2.99667 9.55833; 403 22 13.4933 15.16;
404 22 13.4933 9.55833; 405 25.2967 13.4933 9.55833; 406 25.2967 13.4933 15.16;

MEMBER INCIDENCES
5 13 14; 6 14 15; 13 22 161; 26 5 13; 27 6 14; 28 7 15; 34 13 390; 35 14 22;
36 15 23; 43 22 358; 44 23 359; 50 45 46; 51 46 47; 58 54 162; 71 37 45;
72 38 46; 73 39 47; 79 45 389; 80 46 54; 81 47 55; 88 54 360; 89 55 361;
95 77 402; 103 86 163; 116 69 77; 117 70 78; 118 71 79; 124 77 388; 125 78 86;
126 79 87; 133 86 362; 134 87 363; 140 109 399; 148 118 164; 161 101 109;
162 102 110; 163 103 111; 169 109 357; 170 110 118; 171 111 119; 178 118 364;
179 119 365; 185 13 45; 186 14 46; 187 15 47; 194 22 54; 195 23 55; 209 45 77;
210 46 78; 211 47 79; 218 54 86; 219 55 87; 233 77 109; 234 78 110; 235 79 111;
242 86 118; 243 87 119; 295 153 157; 296 154 158; 298 155 391; 302 157 22;
303 158 54; 304 157 158; 305 159 86; 306 158 159; 308 391 392; 309 161 165;
310 162 166; 311 161 162; 312 163 167; 313 162 163; 314 164 168; 315 163 164;
316 165 23; 317 166 55; 318 165 166; 319 167 87; 320 166 167; 321 168 119;
322 167 168; 544 366 370; 545 370 358; 546 358 374; 547 374 378; 548 378 359;

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595 375 360; 596 360 371; 597 371 367; 598 367 386; 599 385 368; 600 368 372;
601 372 393; 602 362 376; 603 376 380; 604 380 363; 605 365 381; 606 381 377;
607 377 364; 608 364 394; 611 391 159; 613 393 362; 615 393 394; 622 394 392;
623 399 110; 624 392 399; 630 393 391; 631 402 78; 632 391 402; 635 402 399;
636 394 382; 637 382 403; 638 394 406; 639 385 404; 640 393 405; 641 404 405;
642 403 406; 643 405 406; 644 404 403; 645 118 392; 646 392 357; 647 390 387;
648 386 389; 649 385 388; 650 382 357; 651 79 78; 652 111 110;

DEFINE MATERIAL START
ISOTROPIC CONCRETE
E 2.17185e+007
POISSON 0.17
DENSITY 23.5616
ALPHA 1e-005
DAMP 0.05
END DEFINE MATERIAL

MEMBER PROPERTY AMERICAN
26 TO 28 34 TO 36 43 44 161 TO 163 169 TO 171 178 179 647 -
650 PRIS YD 0.45 ZD 0.3
72 80 88 117 125 133 PRIS YD 0.4 ZD 0.4
5 6 50 51 PRIS YD 0.5 ZD 0.3
185 TO 187 209 TO 211 233 TO 235 635 PRIS YD 0.5 ZD 0.3
95 140 623 631 651 652 PRIS YD 0.6 ZD 0.3
MEMBER PROPERTY AMERICAN
304 306 308 311 313 315 318 320 322 555 556 561 TO 566 570 571 573 574 -
615 PRIS YD 0.45 ZD 0.25
194 195 218 219 242 243 558 TO 560 567 TO 569 577 579 581 583 585 -
587 PRIS YD 0.45 ZD 0.25
13 58 295 296 302 303 309 310 316 317 544 TO 548 588 589 592 TO 597 -
598 PRIS YD 0.45 ZD 0.25
103 148 298 305 312 314 319 321 590 599 TO 608 611 613 636 645 -
646 PRIS YD 0.5 ZD 0.25
MEMBER PROPERTY AMERICAN
622 624 630 632 PRIS YD 0.4 ZD 0.25
MEMBER PROPERTY AMERICAN
641 TO 644 PRIS YD 0.35 ZD 0.25
637 TO 640 PRIS YD 0.25 ZD 0.25
MEMBER PROPERTY AMERICAN
71 73 79 81 89 116 118 124 126 134 648 649 PRIS YD 0.45 ZD 0.3
CONSTANTS
MATERIAL CONCRETE ALL
```

```
SUPPORTS
5 TO 7 37 TO 39 69 TO 71 101 TO 103 FIXED

LOAD 1 DL
FLOOR LOAD
YRANGE 10 11 FLOAD -5
YRANGE 6 7 FLOAD -5.44
SELFWEIGHT Y -1
MEMBER LOAD
544 TO 548 558 TO 560 579 581 592 605 TO 608 641 TO 644 LIN Y -2.5 -2.5
5 6 13 95 140 148 185 187 195 209 211 219 233 235 243 295 298 302 308 309 -
314 316 321 577 583 585 587 588 590 599 600 615 623 635 636 645 646 -
652 LIN Y -6 -6
FLOOR LOAD
YRANGE 13 14 FLOAD -5
LOAD 2 LL
FLOOR LOAD
YRANGE 10 11 FLOAD -1.5
YRANGE 13 14 FLOAD -0.75
YRANGE 6 7 FLOAD -3
LOAD 3 SLZ
JOINT LOAD
358 TO 361 363 TO 365 372 376 382 385 TO 387 FZ 3
22 23 54 55 86 87 118 119 357 388 TO 390 FZ 14
LOAD 4 SLX
JOINT LOAD
358 TO 361 363 TO 365 372 376 382 385 TO 387 FX 3
22 23 54 55 86 87 118 119 357 388 TO 390 FX 14

LOAD COMB 5 DL+LL
1 1.0 2 1.0
LOAD COMB 6 DL+SLZ
1 1.0 3 1.0
LOAD COMB 7 DL-SLZ
1 1.0 3-1.0
LOAD COMB 8 DL+SLX
1 1.0 4 1.0
LOAD COMB 9 DL-SLX
1 1.0 4-1.0
LOAD COMB 10 0.75(DL+.5LL+SLZ)
1 0.75 2 0.375 3 0.75
LOAD COMB 11 0.75(DL+0.5LL-SLZ)
1 0.75 2 0.375 3-0.75
LOAD COMB 12 0.75(DL+.5LL+SLX)
1 0.75 2 0.375 4 0.75
LOAD COMB 13 0.75(DL+.5LL-SLX)
1 0.75 2 0.375 4-0.75
```

```
LOAD COMB 14 1.5DL+1.5LL
1 1.5 2 1.5
LOAD COMB 15 1.5DL+1.5SLZ
1 1.5 3 1.5
LOAD COMB 16 1.5DL-1.5SLZ
1 1.5 3-1.5
LOAD COMB 17 1.5DL+1.5SLX
1 1.5 4 1.5
LOAD COMB 18 1.5DL-1.5SLX
1 1.5 4-1.5
LOAD COMB 19 1.2(DL+LL+SLZ)
1 1.2 2 1.2 3 1.2
LOAD COMB 20 1.2(DL+LL-SLZ)
1 1.2 2 1.2 3-1.2
LOAD COMB 21 1.2(DL+LL+SLX)
1 1.2 2 1.2 4 1.2
LOAD COMB 22 1.2(DL+LL-SLX)
1 1.2 2 1.2 4-1.2

PERFORM ANALYSIS PRINT ALL
FINISH
```

8.5.3 Foundation design

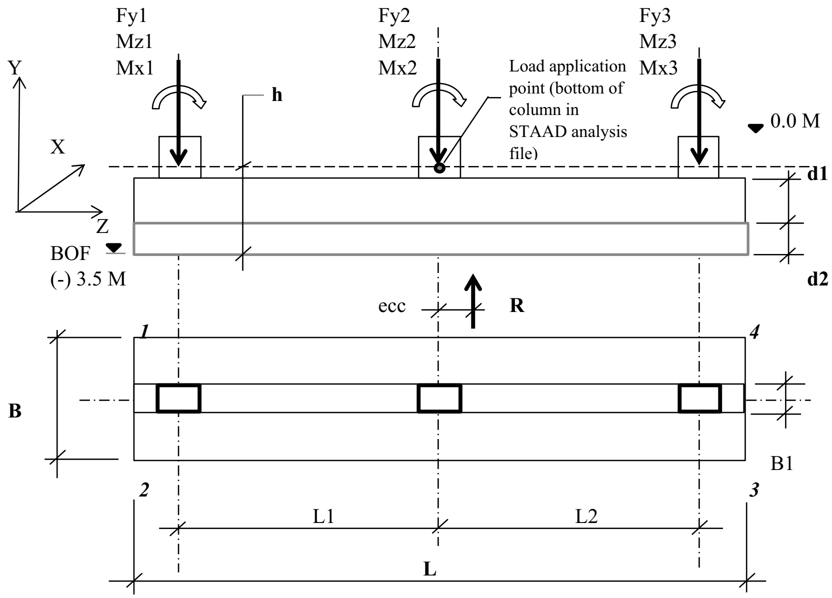


FIGURE 8.2 Strip foundation F1

Design parameters:

L1 =	5.5 m	h =	0.85 m	γ soil =	18 kN/m^3
L2 =	5.5 m	d1 =	0.6 m	γ conc =	24 kN/m^3
L =	12 m	d2 =	0.25 m		
B =	2 m				
B1 =	0.45 m				

Net safe bearing pressure, SBP = 61 kN/m^2

Gross bearing pressure, GBP = 76 kN/m^2

Bottom of foundation, BOF = (–) 3.5 M

Self-weight of foundation =	222 kN	cover	40 mm
Earth backfill =	201 kN	fck	25 Mpa
	423 kN	fy	415 Mpa

Column reactions:

Fy1	382.2	kN
Fx1	15.92	kN
Fz1	–3.26	kN
Mz1	–15.3	kNm
Mx1	–3.13	kNm

Fy2	546.5	kN
Fx2	–14.8	kN
Fz2	–1.31	kN
Mz2	14.44	kNm
Mx2	–1.4	kNm

Fy3	262.2	kN
Fx3	–1.1	kN
Fz3	–3.12	kN
Mz3	1.292	kNm
Mx3	–3.22	kNm

Reaction transferred at BOF, i.e., "h" m below TOP

h= 0.85

Fy1	382.2	kN
Fx1	15.92	kN
Fz1	–3.26	kN
Mz1	–1.79	kNm
Mx1	–5.89	kNm

h= 0.85

Fy2	546.5	kN
Fx2	–14.8	kN
Fz2	–1.31	kN
Mz2	1.85	kNm
Mx2	–2.51	kNm

h= 0.85

Fy3	262.2	kN
Fx3	–1.1	kN
Fz3	–3.12	kN
Mz3	0.36	kNm
Mx3	–5.86	kNm

Mx' = R.e

R = 383.23 + 546.5 + 262.19 + 222 + 201 = 1614 kN including self-weight and earthfill

R = 383.23 + 546.5 + 262.19 = 1191 kN without self-weight and earthfill

Eccentricity = [(546.5 × 5.5 + 262.19 × (5.5 + 5.5) + (–5.89) + (–2.51) + (–5.86) + 222 × 5.5 + 201 × 5.5) / 1614] –5.5

= (–) 0.42 m from center line L/6 = 12 / 6 =2 m from center line of base.

Calculation of soil bearing pressure:

R = 1614 kN
Mx' (= R×e) = 1614 × (−0.42) = −678 kNm
Mz = −1.79 + 1.85+ 0.36 = 0.42 kNm
A = 12 × 2 = 24 m^2
Zx = $(2 \times 12^2) / 6$ = 48 m^3
Zz = $(12 \times 2^2) / 6$ = 8 m^3
Gross bearing pressure:

corner 1	(1614 / 24) − (−677.88 / 48) + (0.42 / 8) =	81.43 kN/m^2
corner 2	(1614 / 24) − (−677.88 / 48) − (0.42 / 8) =	81.32 kN/m^2
corner 3	(1614 / 24) + (−677.88 / 48) − (0.42 / 8) =	53.08 kN/m^2
corner 4	(1614 / 24) + (−677.88 / 48) + (0.42 / 8) =	53.18 kN/m^2
	Average pressure on soil =	67.25 kN/m^2

Gross bearing pressure = 60.5 + 18 × 0.85 = 75.8 kN/m^2 OK.

Net upward pressure = 67.25 − (423 / 24) = 49.63 kN/m^2 OK.

Foundation slab design:

Load factor = 1.5
Provide slab reinforcement at the bottom layer:
Along X – Transv bars 12 diameter @ 150 mm c/c. (Ast = 754 mm^2 /m)
Along Z – long bars 12 diameter @ 200 mm c/c. (Ast = 566 mm^2 /m)

Design check:
Transverse bars: alongX
Projection of slab = 0.775 m
maximum projection = 0.775 m
Moment at face of column = $49.63 \times 0.775^2 / 2$ = 14.90 kNm /m

d reqd= 111 mm dprov = 210 mm **Ok.**
M_u /bd^2 = 0.51
pt = 0.20 M25
Ast reqd. = 420 mm^2/m width
Ast provided = 754 mm^2/m width **Ok.** pt = 0.359 %
Ast min = 252 mm^2/m width
τc.b.d = 193 kN.
Vs = 42 kN < τc.b.d **Ok.**

Long bars: alongZ

pt =	0.20 M25	
Ast reqd. =	420 mm^2/m width	
Ast provided =	566 mm^2/m width	**Ok.**

Shear at critical plane = 49.63 × (0.775 – 0.210) = 28 kN

Shear capacity = τc b d = 0.39 × 1000 × 210 / 1000 = 82 kN

Safe

Foundation beam design

B =	0.45 m	fck =	25 MPa
D =	0.85 m	fy =	415 MPa
Cover =	40 mm	B1 =	2 m.

Net SBP upward = 49.63 kN/m^2

Span = **5.5** m.

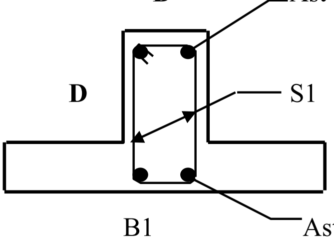


FIGURE 8.3 Cross section of F1

Design forces:

At span		Factored load		Load factor = 1.5
Mspan =	300 kNm	Mu span =	450	
Msupport =	300 kNm	Mu supp =	450	
Fsh =	231kN	F supp =	346	

Reinforcement:

At mid span			**Ast**
Top	6 nos	20 Dia	1884 mm^2
Bot	6 nos	20 dia	1884 mm^2
Stirrup	12 diameter @	200 mm c/c.	(Asv = 226 mm^2 /set)
At support			**Ast**
Top	6 nos	20 diameter +	1884 mm^2
	0 nos	20	
Bot	6 nos	20 dia	1884 mm^2
Stirrup	2 legged	12 diameter @	200 mm c/c. (Asv = 226 mm^2 /set)

Design check:

M_U limit = 1019 kNm Singly reinforced Mulim/bd^2= 3.45
Span moment = 450 kNm
dprov = 810 mm
M_u /bd^2 = 1.53

pt =	0.47	(M25)		pc =	0.2	(M25)
Ast reqd. =	1699	mm^2/m width		Ast reqd. =	729	mm^2/m width
Ast provided =	1884	mm^2/m width	OK	Ast provided =	1884	mm^2/m width	Okay
Ast min =	765	mm^2/m width		Ast min =	765	mm^2/m width

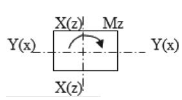
Span moment = 450 kNm M_U limit = 1019 kNm Singly reinforced
dprov = 810 mm
M_u /bd^2 = 1.53

pt =	0.47	(M25)		pc =	0.2	(M25)	
Ast reqd. =	1699	mm^2/m width		Ast reqd. =	729	mm^2/m width	
Ast provided =	1884	mm^2/m width	Okay	Ast provided =	1884	mm^2/m width	Okay
Ast min =	765	mm^2/m width		Ast min =	765	mm^2/m width	

Shear check:

End shear, V= 346 kN
τc.b.d = 164 kN
Vs = 182 kN (V- τc.b.d)
Vus = 331 kN (from shear re-bars)
τc.b.d + Vus = 164 + 331 = 495 kN > V; Safe

8.5.4 Member design



8.5.4.1 Design of column [IS 456: 2000; SP-16]

Column grid: A & D – 1–2–3

Column dimensions

Column Mkd	Location		Fdn	Size		Unsuppt length			
			Mkd	D	B	lex	ley	lex/D	ley/B
				m	m	m	m		
C1				0.45	0.3	3.8	2.2	8.4	7.3

Column Reinforcement

As1	3	nos	20	mm diameter per face
As2	2	nos	20	mm diameter per face

As = 2(As1 + As2) = 3140 mm^2 p = 2.3 %

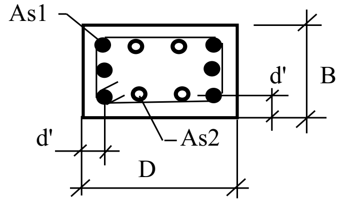


FIGURE 8.4 Column section

Concrete grade = M25 fck = 25 N/mm^2 fy = 415 N/mm^2
Clear cover = 40 mm Ac= 131860 mm^2 d' = 50 mm
Puz = 0.45 fck Ac + 0.75 fy As = 2461 kN

Design of columns [As per SP-16 and IS: 456]

TABLE 8.1
Column load from STAAD analysis output file (factored load)

SL no		Beam	L/C	Node	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
1	Max Fx	162	14 1.5DL+1.5LL	102	820	22	−2	0	2	22
2	Min Fx	179	22 1.2(DL+LL−SLX)	365	87	22	−14	0	−27	−39
3	Max Fy	162	17 1.5DL+1.5SLX	102	735	50	−2	0	3	70
4	Min Fy	161	18 1.5DL−1.5SLX	101	573	−47	−6	0	7	−66
5	Max Fz	36	16 1.5DL−1.5SLZ	15	282	14	29	0	−50	21
6	Min Fz	171	15 1.5DL+1.5SLZ	111	278	10	−29	0	51	10
7	Max Mx	170	15 1.5DL+1.5SLZ	110	500	9	−26	1	44	24
8	Min Mx	169	15 1.5DL+1.5SLZ	109	383	−26	−28	−1	50	−55
9	Max My	36	16 1.5DL−1.5SLZ	23	265	14	29	0	57	−32
10	Min My	171	15 1.5DL+1.5SLZ	119	261	10	−29	0	−58	−28
11	Max Mz	169	18 1.5DL−1.5SLX	357	375	−43	−12	0	−25	74
12	Min Mz	169	18 1.5DL−1.5SLX	109	393	−43	−12	0	19	−86

TABLE 8.2
Design load for columns

Sl no	Load case nos. and combinations [As per STAAD load comb case]					Axial*	Moment*	
						Pu	Mz	My
						kN	kNm	kNm
1	14 1.5 DL+1.5LL					819.7	21.66	2.103
2	22 1.2 (DL+LL−SLX)					86.57	−38.9	−27.1
3	17 1.5 DL+1.5SLX					734.9	69.76	2.556

(*Continued*)

TABLE 8.2 (*Continued*)
Design load for columns

Sl no	Load case nos. and combinations [As per STAAD load comb case]					Axial* Pu kN	Moment* Mz kNm	My kNm
4	18 1.5 DL−1.5SLX					572.8	−66.3	7.21
5	16 1.5 DL−1.5SLZ					282.4	20.66	−50.2
6	15 1.5 DL+1.5SLZ					278.5	10.38	51.28
7	15 1.5 DL+1.5SLZ					500.2	23.5	44.08
8	15 1.5 DL+1.5SLZ					383	−54.8	49.88
9	16 1.5 DL−1.5SLZ					264.5	−32	56.68
10	15 1.5 DL+1.5SLZ					260.6	−28.2	−57.7
11	18 1.5 DL−1.5SLX					374.7	74.29	−24.6
12	18 1.5 DL−1.5SLX					392.6	−85.9	19.28

[* Note: Pu = Fx; Mux = Mz; Muy = My]

TABLE 8.3
Design of column – compression member subject to bi-axial bending

			About major axis					About minor axis				
						Chart 44		d′/B	Chart 46			
L/C	Pu/ Puz	a_n	d′/D	p/fck	Pu/ fckbD	Mux / $fckBD^2$	Mu = Muz		Muy / $fckDB^2$	Muy = Muy	Stress Interaction factor	Remarks
							kNm			kNm		
1	0.33	1.250	0.1	0.09	0.24	**0.14**	212.6	0.2	**0.12**	121.5	**0.1**	Safe.
2	0.04	1.000	0.1	0.09	0.03	**0.13**	197.4	0.2	**0.11**	106.3	**0.5**	Safe.
3	0.30	1.167	0.1	0.09	0.22	**0.14**	212.6	0.2	**0.12**	116.4	**0.3**	Safe.
4	0.23	1.083	0.1	0.09	0.17	**0.14**	212.6	0.2	**0.12**	116.4	**0.3**	Safe.
5	0.11	1.000	0.1	0.09	0.08	**0.14**	212.6	0.2	**0.11**	111.4	**0.5**	Safe.
6	0.11	1.000	0.1	0.09	0.08	**0.14**	212.6	0.2	**0.11**	111.4	**0.5**	Safe.
7	0.20	1.083	0.1	0.09	0.15	**0.14**	212.6	0.2	**0.12**	116.4	**0.4**	Safe.
8	0.16	1.000	0.1	0.09	0.11	**0.14**	212.6	0.2	**0.13**	126.6	**0.7**	Safe.
9	0.11	1.000	0.1	0.09	0.08	**0.14**	212.6	0.2	**0.11**	111.4	**0.7**	Safe.
10	0.11	1.000	0.1	0.09	0.08	**0.14**	212.6	0.2	**0.11**	111.4	**0.7**	Safe.
11	0.15	1.000	0.1	0.09	0.11	**0.14**	212.6	0.2	**0.12**	116.4	**0.6**	Safe.
12	0.16	1.000	0.1	0.09	0.12	**0.14**	212.6	0.2	**0.12**	117.5	**0.6**	Safe.

Stress Interaction factor:

$(Mux/Mux1)^{\alpha n} + (Muy/Muy1)^{\alpha n} <= 1$

Or,

$(Mz/Muz)^{\alpha n} + (My/Muy)^{\alpha n} <= 1$

8.5.4.2 Design of Grade Beam [IS 456: 2000; SP-16]

GB1 [R3] 500 × 250

B =	0.25 m.	fck =	25 Mpa
D =	0.5 m.	fy =	415 Mpa
cover =	40 mm.		
Span =	5.5 m.		

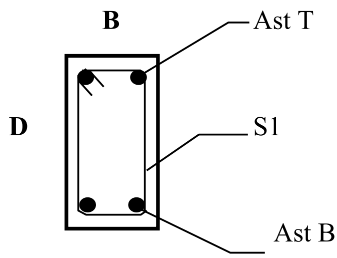


FIGURE 8.5 Grade Beam

Design forces: Load factor 1 [factored load]

Mspan =	73 kNm	Mu span =	73 kNm
Msupport =	87 kNm	Mu supp =	87 kNm
Fsh =	60 kN	F supp =	60 kN

Reinforcement:

At mid span			**Ast**
Top	2 Nos	16 dia	401.92 mm^2.
Bot	3 Nos	16 dia	602.88 mm^2.
Stirrup	8 diameter @ 250 mm c/c. (Asv =		100.56 mm^2 /set)
At support			**Ast**
Top	2 nos	16 diameter +	602.88 mm^2.
	1 nos	16	
Bot	2 nos	16 dia	401.92 mm^2.
Stirrup	2 legged	8 diameter @	100 mm c/c. (Asv = 100.56 mm^2 / set).

Design check:

M_U limit =	183 kNm	Singly reinforced	Mulim/bd^2=	3.45
Span moment =	72.5 kNm			
dprov =	460 mm			
M_u /bd^2 =	1.37			
pt =	0.42 (M25)	pc =	0.2 (M25)	
Ast reqd. =	480 mm^2/m width	Ast reqd. =	230 mm^2/m width	
Ast provided =	603 mm^2/m OK. width.	Ast provided =	402 mm^2/m width.	Okay.
Ast min =	250 mm^2/m width	Ast min =	250 mm^2/m width	

Support moment =	87 kNm	M_U limit =	183 kNm Singly reinforced	
dprov =	460 mm			
M_u /bd^2 =	1.64			
pt =	0.50 (M25)	pc =	0.2 (M25)	
Ast reqd. =	574 mm^2/m width	Ast reqd. =	230 mm^2/m width	
Ast provided =	603 mm^2/m OK width	Ast provided =	402 mm^2/m width	Okay
Ast min =	250 mm^2/m width	Ast min =	250 mm^2/m width	

Shear check:

End shear, V=	60 kN	
τc.b.d =	52 kN	
Vs =	8 kN	(V- τc.b.d)
Vus =	167 kN	(from shear re-bars)
τc.b.d + Vus = 52 + 167 =	219 kN	> V; Safe

Design of other grade beams have been done following the same procedure as above.

8.5.4.3 Design of the floor beam

FB23 [R8] 450 × 250

LC	11

B	0.25 m	fck	25 Mpa
D	0.45 m	fy	415 Mpa
cover	40 mm		
Span =	5.6 m.		

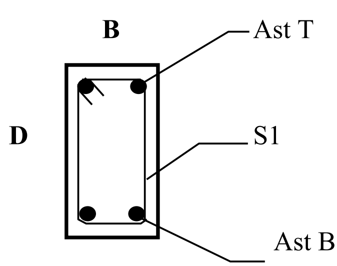


FIGURE 8.6 Floor beam

Design forces: Load factor 1 [factored load]

Mspan =	141 kNm	Mu span =	141 kNm
Msupport =	188 kNm	Mu supp =	188 kNm
Fsh =	163 kN	F supp =	163 kN

Reinforcement:

At mid span			**Ast**	
Top	2 Nos	20 dia	628	mm^2.
Bot	3 Nos	20 dia	1256	mm^2.
Stirrup	8 diameter @ 250 mm c/c. (Asv =		101 mm^2 /set)	
At support			**Ast**	
Top	3 nos	20 diameter +	1570	mm^2.
	2 nos	20		
Bot	3 nos	20 dia	942	mm^2.
Stirrup	2 legged	8 diameter @	150 mm c/c. (Asv = 101	mm^2 / set).

Design check:

M_U limit =	145 kNm	Doubly reinforced		Mulim/bd^2=	3.45	
Span moment =	141 kNm					
dprov =	410 mm					
M_u /bd^2 =	3.36					
pt =	1.15 (M25)		pc =	0.2 (M25)		
Ast reqd. =	1182 mm^2/m width		Ast reqd. =	205 mm^2/m width		
Ast provided =	1256 mm^2/m width.	Okay.	Ast provided =	628 mm^2/m width.	Okay.	
Ast min =	225 mm^2/m width		Ast min =	225 mm^2/m width		

Support moment =	188 kNm	M_U limit =		145 kNm Singly reinforced	
dprov =	410 mm				
M_u /bd^2 =	4.47				
pt =	1.52 (M25)		pc =	0.34 (M25)	
Ast reqd. =	1555 mm^2/m width		Ast reqd. =	350 mm^2/m width	
Ast provided =	1570 mm^2/m width	OK	Ast provided =	942 mm^2/m width	Okay
Ast min =	225 mm^2/m width		Ast min =	225 mm^2/m width	

Shear check:

End shear, V=	163 kN		
$\tau c.b.d$ =	75.85 kN		
Vs =	87.15 kN	(V- $\tau c.b.d$)	
Vus =	99.67 kN	(from shear re-bars)	
$\tau c.b.d$ + Vus = 75.85 + 99.67 =	176 kN		> V; Safe

All other beams in floor and roof are done in the same procedure as shown above.

8.5.4.4 Design of slabs and stairs

Roof slab

UDL1 =	5.1 + 1.5 =	6.5 kN/m^2	DL + LL
Span =	2.25 M		

Moment span = 3.29 kNm
Moment support = 2.74 kNm
R1 = 7.31 kN R2 = 7.31 kN

Try with	**120** mm	thick slab	B	1 m.	fck	25 Mpa
			D	0.12 m.	fy	415 Mpa
			cover	15 mm.	Load factor	1.5

Re-bars	Span	8 diameter @	150 mm c/c.	(Ast =	335 mm^2 /m)
	support	8 diameter @	150 mm c/c.	(Ast =	335 mm^2 /m)

Design check:

M_U limit =	38 kNm	Singly reinforced		Mulim/bd^2=	3.45
Span moment =	5 kNm				
dprov =	105 mm				
M_u /bd^2 =	0.45				
pt =	0.20 (M25)		pc =	0.20 (M25)	
Ast reqd. =	240 mm^2/m width		Ast reqd. =	240 mm^2/m width	
Ast provided =	335 mm^2/m width.	OK.	Ast provided =	335 mm^2/m width.	OK.
Ast min =	144 mm^2/m width		Ast min =	144 mm^2/m width	

Support moment =	4.1 kNm	M_U limit =	38 kNm	Singly reinforced	
dprov =	105 mm				
M_u/bd^2 =	0.37				
pt =	0.2 (M25)		pc =	0.20 (M25)	
Ast reqd. =	240 mm^2/m width		Ast reqd. =	240 mm^2/m width	
Ast provided =	335 mm^2/m width	Okay	Ast provided =	335 mm^2/m width	Okay
Ast min =	144 mm^2/m width		Ast min =	144 mm^2/m width	

Shear check:

End shear, V=	7.31 kN	
$\tau c.b.d$ =	34.7 kN	> V; Safe

Shear check is generally not necessary for standard loading.

Floor slab

UDL1 = 5.44 + 3 = 8.44 kN/m^2 DL + LL
Span = 2.25 M

Moment span = 4.27 kNm
Moment support = 2.74 kNm
R1 = 9.50 kN R2 = 9.50 kN

Try with	**120** mm	thick slab	B	1 m.	fck	25 Mpa
			D	0.12 m.	fy	415 Mpa
			cover	15 mm.	Load factor	1.5

Re-bars	Span	8 diameter @	150 mm c/c.	(Ast =	335 mm^2 /m)
	support	8 diameter @	150 mm c/c.	(Ast =	335 mm^2 /m)

Design check:

M_U limit =	38 kNm	Singly reinforced	Mulim/bd^2=	3.45
Span moment =	6.41 kNm			
dprov =	105 mm			
M_u/bd^2 =	0.58			
pt =	0.20 (M25)		pc =	0.20 (M25)
Ast reqd. =	240 mm^2/m width		Ast reqd. =	240 mm^2/m width

Ast provided =	335 mm²/m width.	Okay.	Ast provided =	335 mm²/m width.	Okay.
Ast min =	144 mm²/m width		Ast min =	144 mm²/m width	

Support moment =	5.34 kNm	M_U limit =	38 kNm	Singly reinforced
dprov =	105 mm			
M_u/bd^2 =	0.48			
pt =	0.2 (M25)		pc =	0.20 (M25)
Ast reqd. =	240 mm²/m width		Ast reqd. =	240 mm²/m width
Ast provided =	335 mm²/m width Okay		Ast provided =	335 mm²/m width Okay
Ast min =	144 mm²/m width		Ast min =	144 mm²/m width

Shear check:

End shear, V= 9.5 kN

$\tau c.b.d$ = 34.7 kN > V; Safe

Shear check is generally not necessary for standard loading.

Stair case

Steps:	Tread = 250 mm	Riser = 150 mm	
Landing slab	150 mm thick	Waist slab	150 mm thick (on slope)

Load intensity

Waist slab		DL	LL
150 mm	slab	5.09 kN/m²	
40 mm	Floor finish	0.96	
12 mm	Plaster	0.24	
		6.29	5 kN/m²
	Steps	1.88 kN/m²	

Landing slab		DL	LL
150 mm	Slab	3.6 kN/m²	
40 mm	Floor finish	0.96	
12 mm	Plaster	0.24	
		4.8 kN/m²	5 kN/m²

Flight 1

	DL	LL
UDL1	8.17	5.00 kN/m^2
UDL2	4.8	5 kN/m^2

SPAN **3.75** M

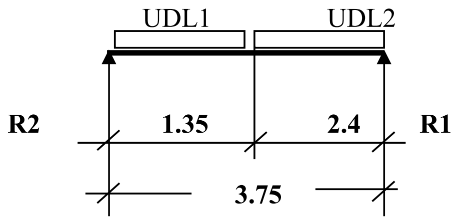


FIGURE 8.7 Stair slab 1

Moment span = 15 kNm
Moment support = 17 kNm
R1 = 17 kN R2 = 5.51 kN

Try with **150** mm	thick slab	B	1 m.	fck	25 Mpa
		D	0.15 m.	fy	415 Mpa
		cover	25 mm.	Load factor	1.5

Reinforcement bars:

Span	Bottom	**12** Diameter @	**150** mm c/c. (Ast =	754 mm^2 /m)	Ok.
	Top	**8** diameter @	**200** mm c/c. (Ast =	251 mm^2 /m)	Ok.
Support	Top	**12** diameter @	**150** mm c/c. (Ast =	754 mm^2 /m)	Ok.
	Bottom	**8** diameter @	**200** mm c/c. (Ast =	251 mm^2 /m)	Ok.

Design check:

M_U limit =	54 kNm	Singly reinforced	Mulim/bd^2=	3.45	
Span moment =	22 kNm				
dprov =	125 mm				
M_u /bd^2 =	1.40				
pt =	0.43 (M25)		pc =	0.20 (M25)	
Ast reqd. =	650 mm^2/m width		Ast reqd. =	300 mm^2/m width	
Ast provided =	754 mm^2/m width.	OK.	Ast provided =	754 mm^2/m width.	Okay.
Ast min =	180 mm^2/m width		Ast min =	180 mm^2/m width	

Support moment = 25 kNm M_U limit = 54 kNm Singly reinforced
dprov = 125 mm
M_u /bd^2 = 1.63

pt =	0.50 (M25)		pc =	0.20 (M25)	
Ast reqd. =	749 mm^2/m width		Ast reqd. =	300 mm^2/m width	
Ast provided =	754 mm^2/m width	OK	Ast provided =	754 mm^2/m width	Okay
Ast min =	180 mm^2/m width		Ast min =	180 mm^2/m width	

Shear check:

End shear, V=	17 kN	
τc.b.d =	54 kN	> V; Safe

Flight 2

Width = 1.2 m

	DL	LL
UDL1	8.17	5.00 kN/m^2
UDL2	4.80	5.00 kN/m^2

SPAN **4.9** M [CL 33.1 IS 456:2000]

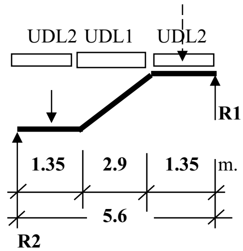


FIGURE 8.8 Stair slab 2

Moment span =	15 kNm			
Moment support =	17 kNm			
R1 =	17 kN	R2 =	5.51 kN	

Try with	**150** mm	thick slab	B	1 m.	fck	25 Mpa
			D	0.15 m.	fy	415 Mpa
			cover	25 mm.	Load factor	1.5

Reinforcement bars:

Span	Bottom	**16** Diameter @	**140** mm c/c. (Ast =	1437 mm^2 /m)
	Top	**8** diameter @	**200** mm c/c. (Ast =	251 mm^2 /m)
Support	Top	**16** diameter @	**140** mm c/c. (Ast =	1437 mm^2 /m)
	Bottom	**8** diameter @	**200** mm c/c. (Ast =	251 mm^2 /m)

Design check:

Load factor =	1.5					
M_U limit =	54 kNm	Singly reinforced		$Mulim/bd^2$=	3.45	
Span moment =	37 kNm					
dprov =	125 mm					
M_u/bd^2 =	2.40					
pt =	0.76 (M25)		pc =	0.20 (M25)		
Ast reqd. =	1143 mm^2/m width		Ast reqd. =	300 mm^2/m width		
Ast provided =	1437 mm^2/m width.	Okay.	Ast provided =	1437 mm^2/m width.	Okay.	
Ast min =	180 mm^2/m width		Ast min =	180 mm^2/m width		

Support moment =	43 kNm	M_U limit =	54 kNm	Singly reinforced	
dprov =	125 mm				
M_u/bd^2 =	2.77				
pt =	0.92 (M25)		pc =	0.20 (M25)	
Ast reqd. =	1374 mm^2/m width		Ast reqd. =	300 mm^2/m width	
Ast provided =	1437 mm^2/m width	Okay.	Ast provided =	1437 mm^2/m width	Okay
Ast min =	180 mm^2/m width		Ast min =	180 mm^2/m width	

Shear check:

End shear, V=	15 kN	
$\tau c.b.d$ =	71 kN	> V; Safe

8.6 CONSTRUCTION DRAWING

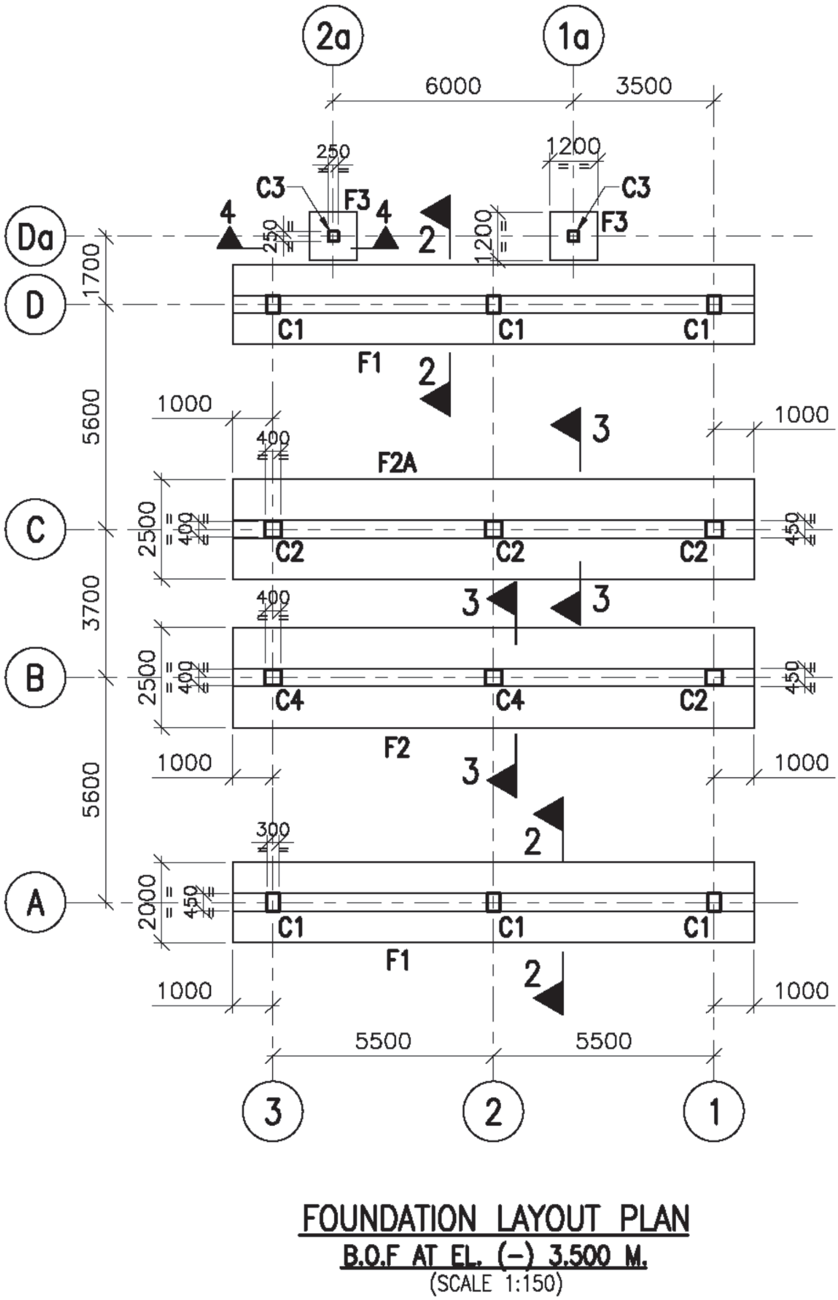


FIGURE 8.9(A) Foundation Layout

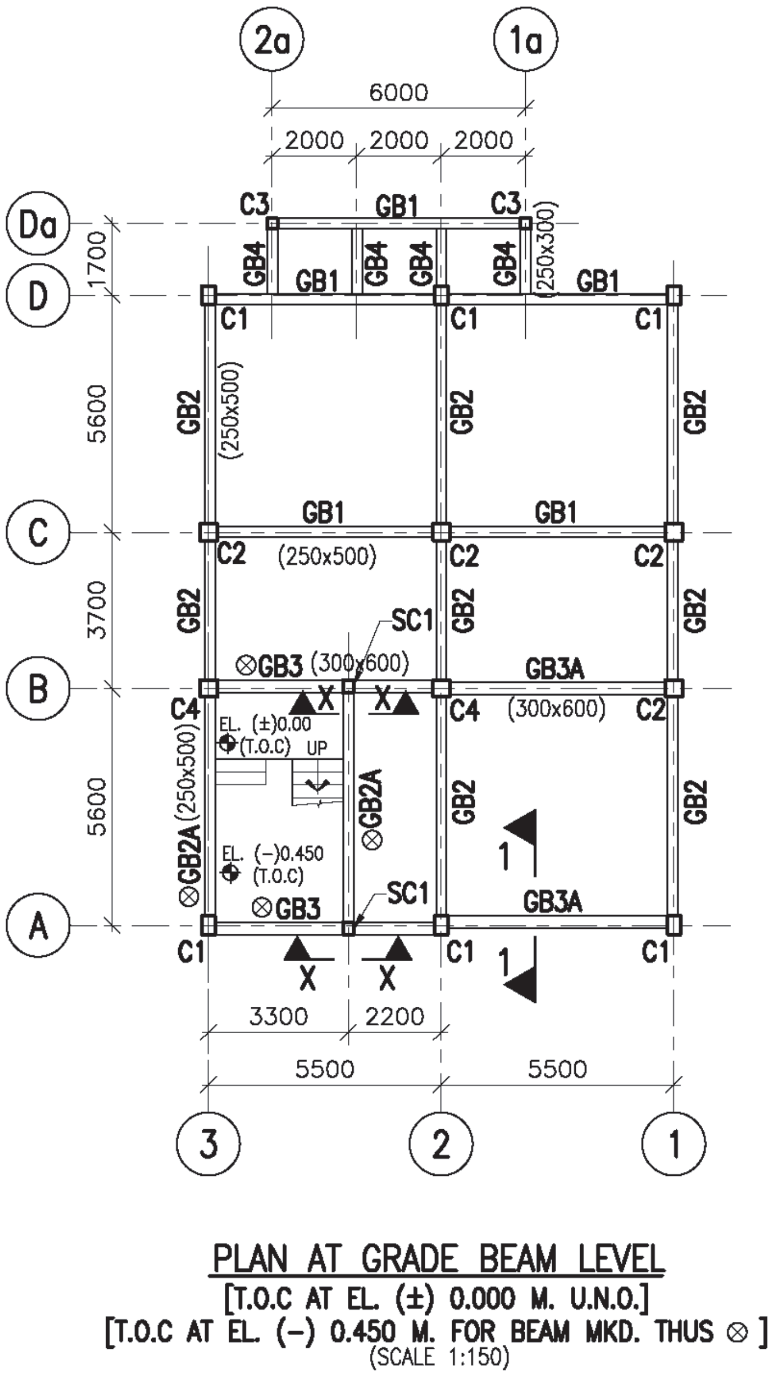


FIGURE 8.9(B) Grade beams

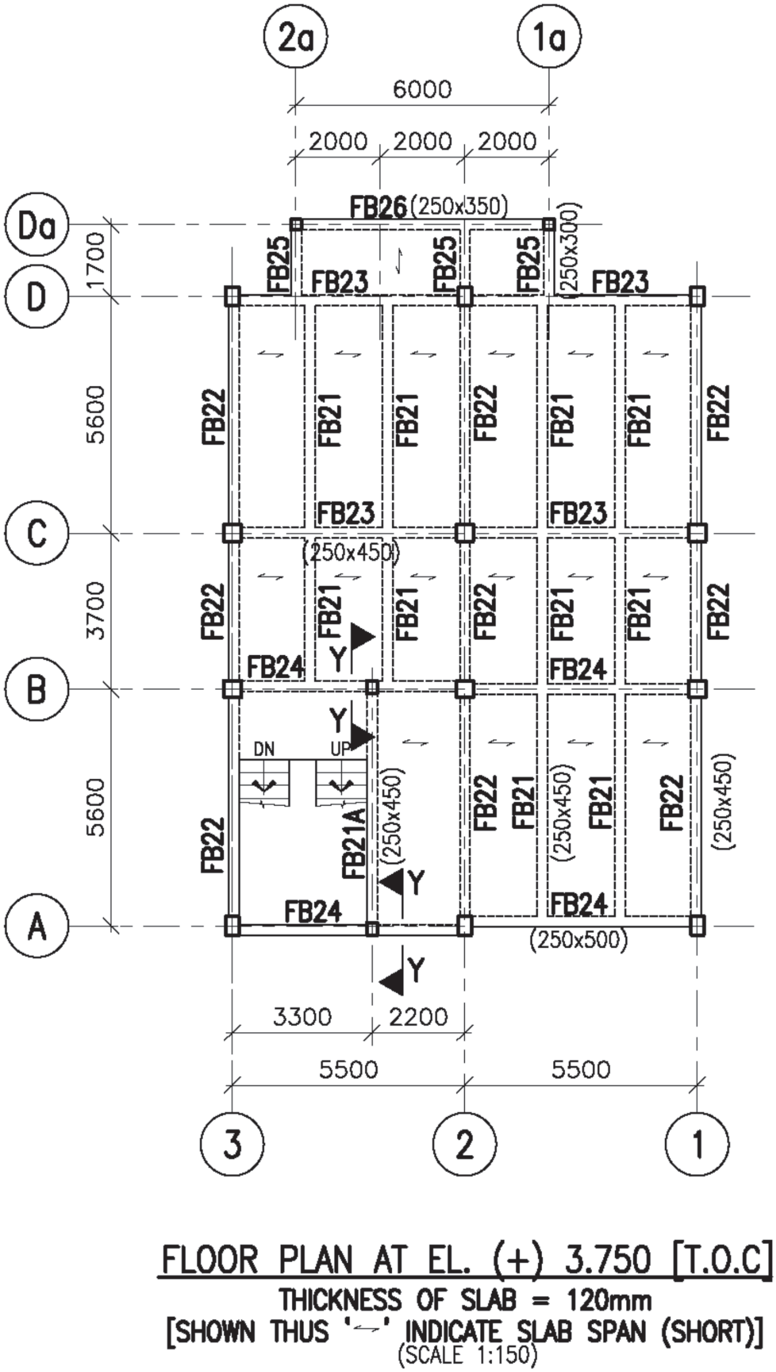


FIGURE 8.9(C) Floor Beams at EL 3.75 M

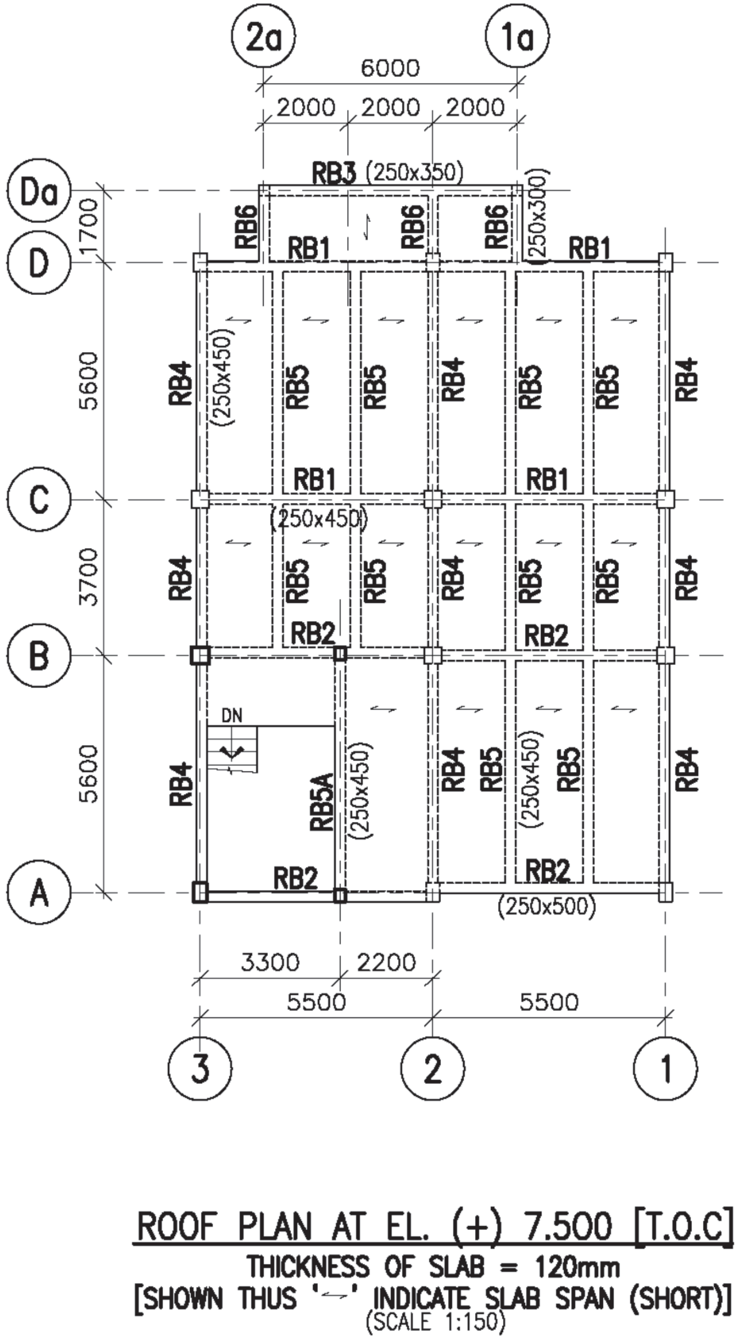


FIGURE 8.9(D) Roof beams at EL 7.5M

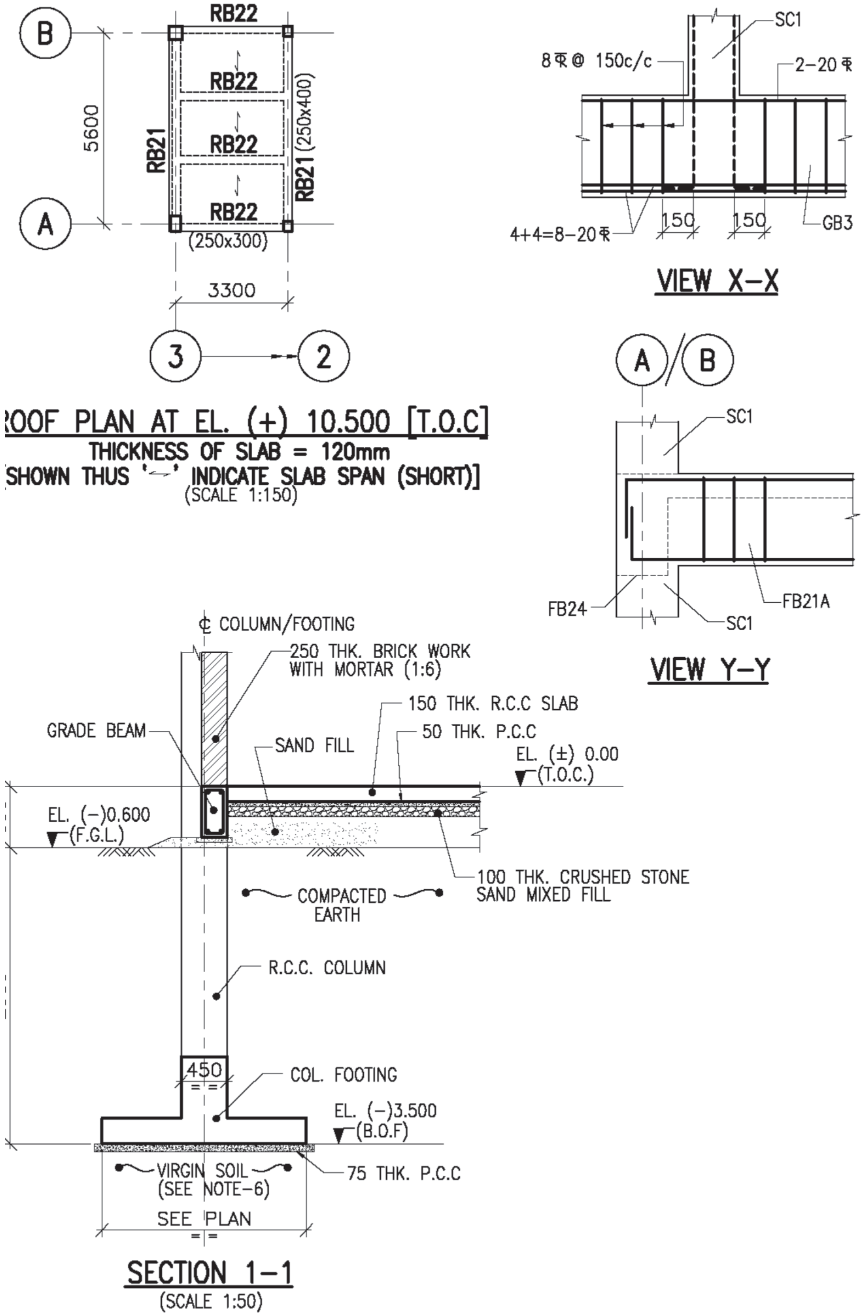
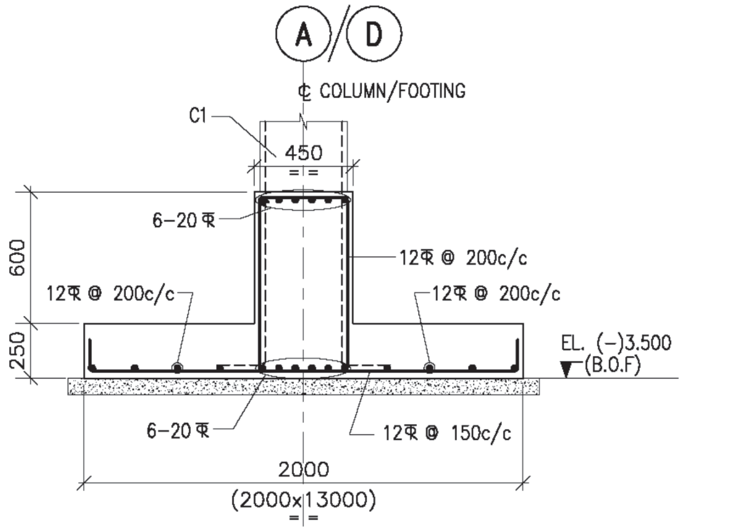
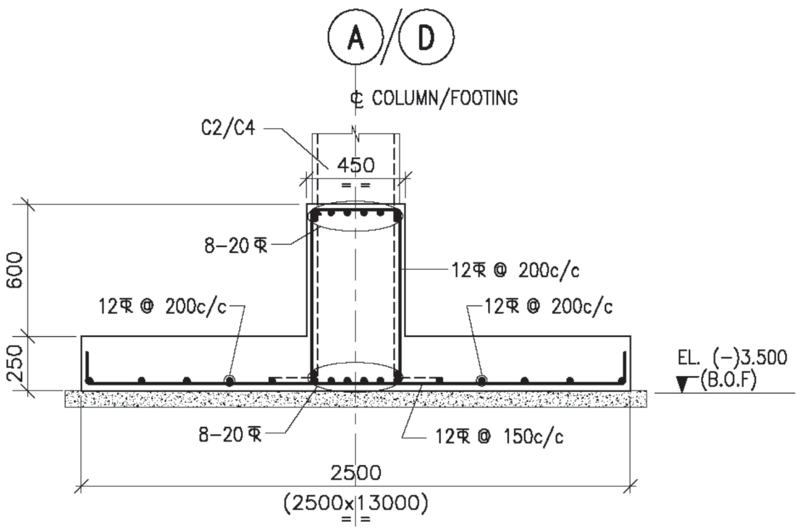


FIGURE 8.9(E) Stair roof and sections



SECTION 2–2
DETAIL OF FOOTING MKD. F1
(SCALE 1:25)



SECTION 3–3
DETAIL OF FOOTING MKD. F2 & F2A
(SCALE 1:25)

FIGURE 8.9(F) Sections of footings

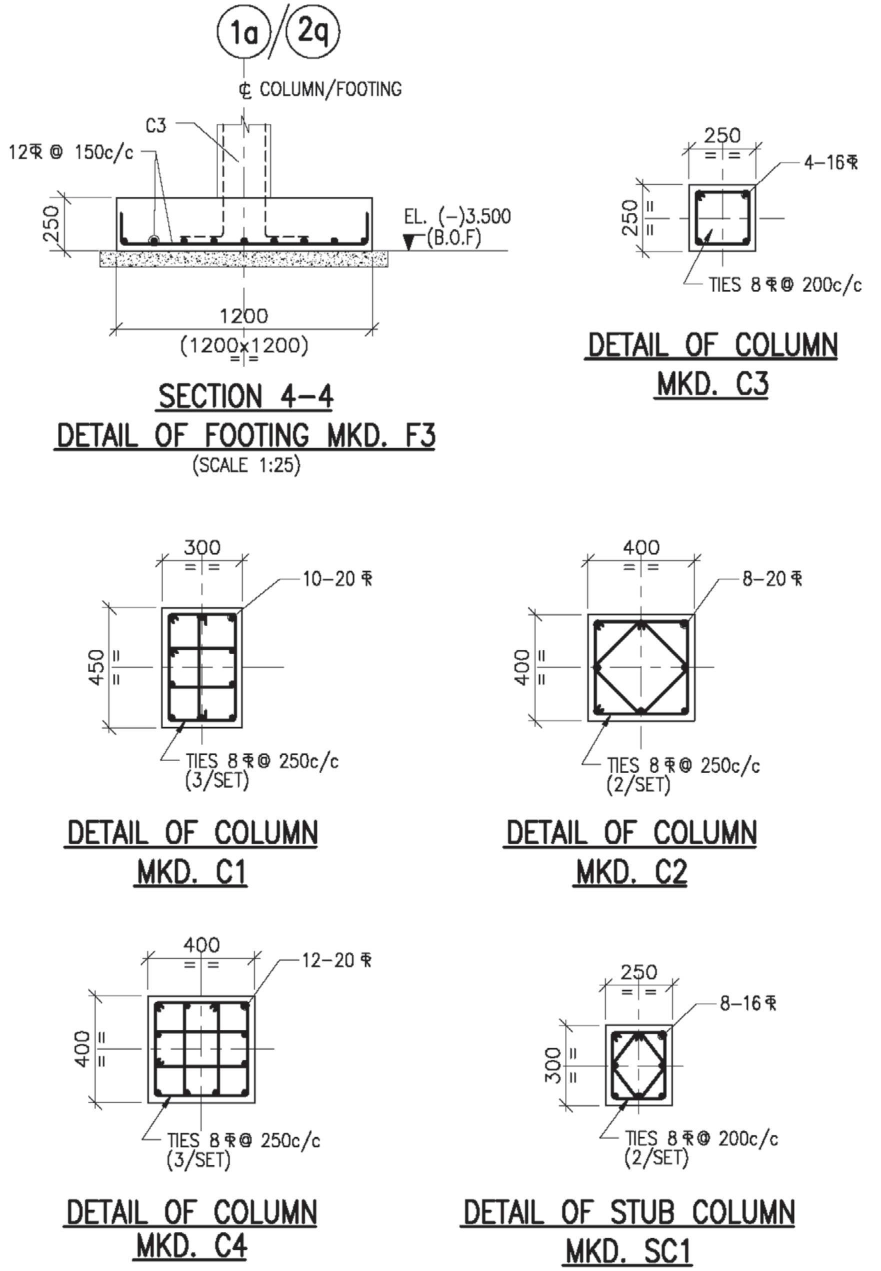


FIGURE 8.9(G) Detail of columns and foundation

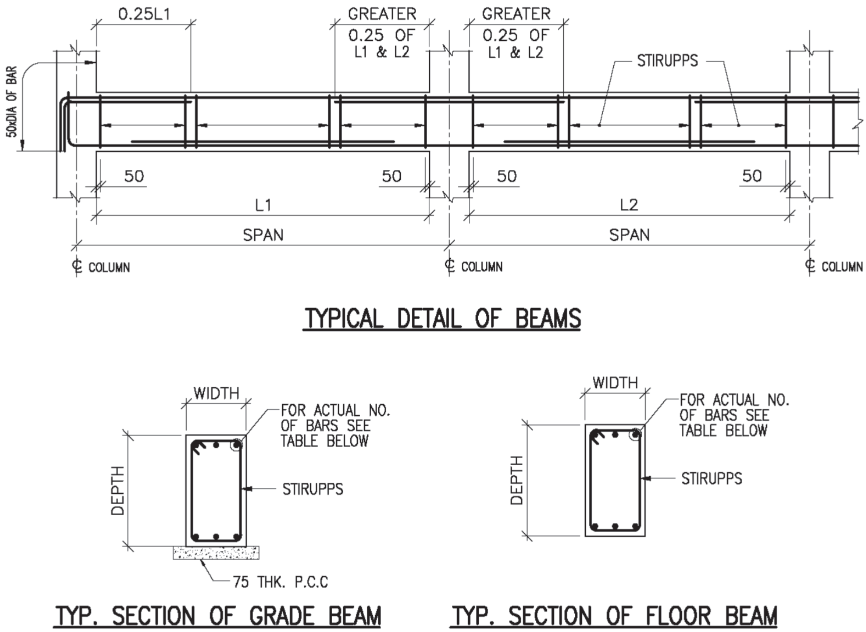


FIGURE 8.9(H) Details of beam reinforcement

S C H E D U L E O F B E A M S

BEAM MKD.	T.O.C (M.)	BEAM SIZE		REINFORCEMENTS						REMARKS
		WIDTH (mm)	DEPTH (mm)	AT SUPPORT			AT MID SPAN			
				TOP	BOT.	STIRRUP	TOP	BOT.	STIRRUP	
GB1	(±)0.000	250	500	3–16ϕ	2–16ϕ	2L–8ϕ@ 100 c/c	2–16ϕ	3–16ϕ	2L–8ϕ@ 250 c/c	
GB2	(±)0.000	250	500	3–16ϕ	2–16ϕ	2L–8ϕ@ 100 c/c	2–16ϕ	3–16ϕ	2L–8ϕ@ 250 c/c	
GB3	(−)0.450	300	600	4–20ϕ	3–20ϕ	2L–8ϕ@ 150 c/c	2–20ϕ	8–20ϕ	2L–8ϕ@ 150 c/c	
GB2A	(−)0.450	250	500	2–16ϕ	2–16ϕ	2L–8ϕ@ 100 c/c	2–16ϕ	3–16ϕ	2L–8ϕ@ 250 c/c	
GB4	(±)0.000	250	300	2–16ϕ	2–16ϕ	2L–8ϕ@ 150 c/c	2–16ϕ	2–16ϕ	2L–8ϕ@ 150 c/c	
GB3A	(±)0.000	300	600	4–20ϕ	3–20ϕ	2L–8ϕ@ 150 c/c	2–20ϕ	4–20ϕ	2L–8ϕ@ 200 c/c	
FB21	(+)3.750	250	450	3–16ϕ	2–16ϕ	2L–8ϕ@ 150 c/c	2–16ϕ	3–16ϕ	2L–8ϕ@ 250 c/c	
FB21A	(+)3.750	250	450	4–16ϕ	2–16ϕ	2L–8ϕ@ 150 c/c	2–16ϕ	4–16ϕ	2L–8ϕ@ 250 c/c	
FB22	(+)3.750	250	450	3–16ϕ	2–16ϕ	2L–8ϕ@ 150 c/c	2–16ϕ	3–16ϕ	2L–8ϕ@ 250 c/c	
FB23	(+)3.750	250	450	5–20ϕ	3–20ϕ	2L–8ϕ@ 150 c/c	2–20ϕ	4–20ϕ	2L–8ϕ@ 250 c/c	
FB24	(+)3.750	250	500	5–20ϕ	3–20ϕ	2L–8ϕ@ 150 c/c	2–20ϕ	4–20ϕ	2L–8ϕ@ 250 c/c	
FB25	(+)3.750	250	300	2–16ϕ	2–16ϕ	2L–8ϕ@ 150 c/c	2–16ϕ	2–16ϕ	2L–8ϕ@ 150 c/c	
FB26	(+)3.750	250	350	2–16ϕ	2–16ϕ	2L–8ϕ@ 150 c/c	2–16ϕ	4–16ϕ	2L–8ϕ@ 250 c/c	

THE ABOVE REINFORCEMENT DETAILS ARE PRELIMINARY AND NOT FOR CONSTRUCTION

FIGURE 8.9(I) Schedule of beam reinforcement bars

SCHEDULE OF BEAMS										
BEAM MKD.	T.O.C (M.)	BEAM SIZE		REINFORCEMENTS						REMARKS
		WIDTH (mm)	DEPTH (mm)	AT SUPPORT			AT MID SPAN			
				TOP	BOT.	STIRRUP	TOP	BOT.	STIRRUP	
RB1	(+)7.500	250	450	4–20ϕ	2–20ϕ	2L–8ϕ@ 150 c/c	2–20ϕ	3–20ϕ	2L–8ϕ@ 250 c/c	
RB2	(+)7.500	250	450	5–20ϕ	3–20ϕ	2L–8ϕ@ 150 c/c	2–20ϕ	3–20ϕ	2L–8ϕ@ 250 c/c	
RB3	(+)7.500	250	350	2–16ϕ	2–16ϕ	2L–8ϕ@ 150 c/c	2–16ϕ	4–16ϕ	2L–8ϕ@ 250 c/c	
RB4	(+)7.500	250	450	4–16ϕ	2–16ϕ	2L–8ϕ@ 150 c/c	2–16ϕ	3–16ϕ	2L–8ϕ@ 250 c/c	
RB5	(+)7.500	250	450	2–16ϕ	2–16ϕ	2L–8ϕ@ 150 c/c	2–16ϕ	2–16ϕ	2L–8ϕ@ 250 c/c	
RB5A	(+)7.500	250	450	3–16ϕ	2–16ϕ	2L–8ϕ@ 150 c/c	2–16ϕ	4–16ϕ	2L–8ϕ@ 250 c/c	
RB6	(+)7.500	250	300	2–16ϕ	2–16ϕ	2L–8ϕ@ 150 c/c	2–16ϕ	2–16ϕ	2L–8ϕ@ 150 c/c	
RB21	(+)10.50	250	400	2–20ϕ	2–20ϕ	2L–8ϕ@ 150 c/c	2–20ϕ	3–20ϕ	2L–8ϕ@ 250 c/c	
RB22	(+)10.50	250	300	2–16ϕ	2–16ϕ	2L–8ϕ@ 150 c/c	2–16ϕ	3–16ϕ	2L–8ϕ@ 150 c/c	
THE ABOVE REINFORCEMENT DETAILS ARE PRELIMINARY AND NOT FOR CONSTRUCTION										

FIGURE 8.9(J) Schedule of roof beams

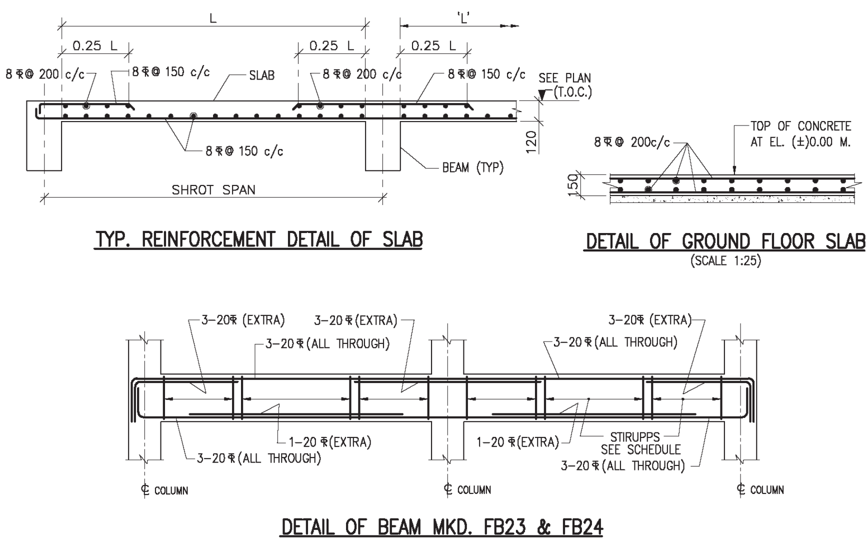


FIGURE 8.9(K) Details of reinforcement in slab and beam

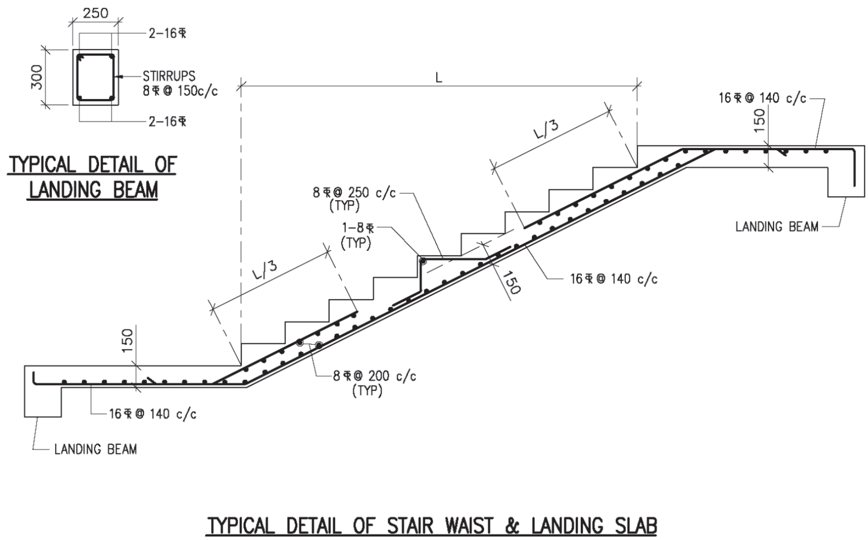


FIGURE 8.9(L) Reinforcement details of staircase

NOTES :

1. ALL DIMENSIONS ARE IN MILLIMETRE AND ELEVATIONS ARE IN METER.
2. ALL CONCRETE WORK SHALL BE DONE IN ACCORDANC WITH IS:456.
3. GRADE OF CONCRETE :
   a) ALL FOUNDATIONS & COLUMNS, BELOW GROUND FLOOR CONTACT WITH SOIL – M25.
   b) SUPER STRUCTURE ABOVE EL. (±)0.00 – M25.
4. REINFORCEMENT BARS SHALL BE Fe 415 (YIELD STRE –415 MPa) CONFORMING TO IS:1786.
5. THE CLEAR COVER TO REINFORCEMENT SHALL BE AS UNDER:
   a) FOUNDATION : 50mm (TOP & SIDES), 75mm (BOTTOM)
   b) COLUMN : 40mm
   c) GRADE BEAM : 30mm
   d) FLOOR BEAM : 30mm
   e) SLAB : 20mm
6. ALL FOUNDATIONS SHALL REST OVER VIRGIN SOIL; IF VIRGIN SOIL IS NOT MET AT FOUNDING LEVEL, THE EXCAVATION TO BE CARRIED OUT TILL THE VERGIN SO IS MET. ADDITIONAL DEPTH TO BE FILLED IN SAND.
7. THE REINFORCEMENT AND MEMBER SIZES GIVEN HERE ARE PRELIMINARY AND NOT FOR CONSTRUCTION.

FIGURE 8.9(M) Standard notes

LEGEND :

1.	F.G.L	: FINISH GRADE LEVEL
2.	B.O.F	: BOTTOM OF FOUNDATION
3.	T.O.C	: TOP OF CONCRETE
4.	P.C.C	: PLAIN CEMENT CONCRETE
5.	R.C.C	: REINFORCEMENT CEMENT CONCRETE
6.	THK.	: THICKNESS
7.	TYP	: TYPICAL
8.	c/c	: CENTRE TO CENTRE
9.	℄	: CENTRE LINE

FIGURE 8.9(M) (*Continued*) Standard notes

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