STRUCTURAL DESIGN OF MASONRY & TIMBER

R - 2021 - AR3402

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AR3402 STRUCTURAL DESIGN OF MASONRY AND TIMBER **L T P/S C**

OBJECTIVES **3 0 0 3**

- To introduce soil mechanics.
- To enable the learning of design of masonry walls.
- To give knowledge to design different timber components in a building.

UNIT 1 SOIL MECHANICS 6

Introduction to Soil Mechanics. Concept of Safe bearing capacity. Types of Soil. Distribution of upward pressure in different types of soil..

UNIT 2 BASIC MASONRY DESIGN 9

Analysis and design of masonry wall with and without openings. Use of nomograms. Code requirements.

UNIT 3 DESIGN OF TIMBER BEAMS 9

Grading of timber. Permissible stresses. Design of solid timber beams. Madras terrace roof design.

UNIT 4 DESIGN OF TIMBER COLUMNS 9

Design of solid timber columns. Built-up Column. Spaced Column.

UNIT 5 ANALYSIS AND DESIGN OF TIMBER TRUSSES 12

Analysis of plane trusses. Introduction to Determinate and Indeterminate plane trusses. Analysis of simply supported and cantilevered trusses by method of joints and method of sections. Analysis and design of timber trusses.

OUTCOME **TOTAL : 45 PERIODS**

- Knowledge about soil mechanics.
- Ability to design structural masonry walls.
- Ability to design timber beams, columns and trusses by applying the code provisions.

TEXT BOOKS

1. P. Dayaratnam, ‘Brick and Reinforced Brick Structures’, CBS Publishers and Distributors Pvt Ltd, 2017.
2. A.S.Arya, ‘Structural Design in Steel, Masonry and Timber’, Nemchand and Bros, Roorkee, 1971.
3. B. C. Punmia, Ashok Kumar Jain , Arun Kumar Jain, , ‘Comprehensive Design of Steel Structures’, Laxmi Publications (P) Ltd, 2014.
4. B. C. Punmia, Ashok Kumar Jain , Arun Kumar Jain , ‘Soil Mechanics and Foundations’, Laxmi Publications, 2017
5. R.K. Bansal, 'A Text Book on Strength of Materials', Laxmi Publications, New Delhi, 2009.

REFERENCE BOOKS

1. National Building Code of India, Part VI, Structural Design,1983.
2. IS 883 – Code of Practice for Design of Structural Timber in Buildings.
3. IS 1905- Code of Practice for Structural use of Unreinforced Masonry-1987.
4. Sp 10- Nomograms for Thickness of Masonry Walls - 1975

UNIT 1

INTRODUCTION TO SOIL MECHANICS

1.1 INTRODUCTION

Soil mechanics is a branch of soil physics and applied mechanics that describes the behavior of soils. It differs from fluid mechanics and solid mechanics in the sense that soils consist of a heterogeneous mixture of fluids (usually air and water) and particles (usually clay, silt, sand, and gravel) but soil may also contain organic solids and other matter. Along with rock mechanics, soil mechanics provides the theoretical basis for analysis in geotechnical engineering, a sub-discipline of civil engineering, and engineering geology, a sub-discipline of geology. Soil mechanics is used to analyze the deformations of and flow of fluids within natural and man-made structures that are supported on or made of soil, or structures that are buried in soils. Example applications are building and bridge foundations, retaining walls, dams, and buried pipeline systems. Principles of soil mechanics are also used in related disciplines such as geophysical engineering, coastal engineering, agricultural engineering, hydrology and soil physics..

1.2 SOIL

ENGINEER	:	It is natural aggregate of mineral grains, loose or moderately cohesive inorganic or organic in nature
GEOLOGIST	:	It is the material in the relative thin zone of the Earth's surface within which roots occur, and which are formed as the products of past surface processes. The rest of the crust is grouped under the term "rock"
AGRICULTURIST	:	It is the substance existing on the surface, which supports plant life

1.2.1 SOIL MECHANICS

Soil Mechanics is a discipline of Civil Engineering involving *the study of soil, its behaviour and application as an engineering material*. Soil Mechanics is the application of laws of mechanics and hydraulics to engineering problems dealing with sediments and other unconsolidated accumulations of solid particles, which are produced by the mechanical and chemical disintegration of rocks, regardless of whether or not they contain an admixture of organic constituents.

1.3 GEOLOGY

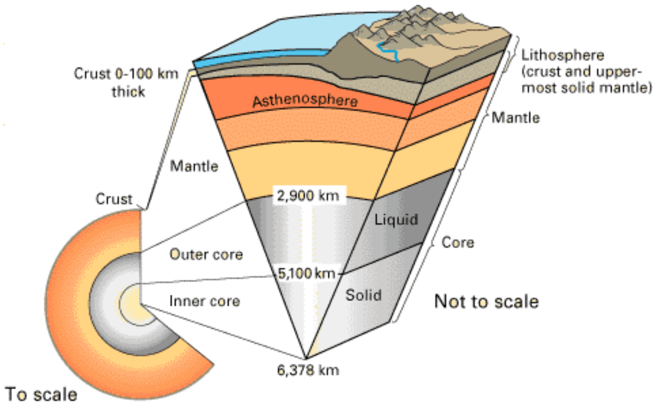
Geology is the science of rocks, minerals, soils and subsurface water, including the study of their formation, structure and behavior.

Engineering Geology is the branch that deals with the application of geologic principles to engineering works

1.3.1 STRUCTURE OF EARTH

The interior of the earth can be divided into 3 different layers.

- Crust
- Mantle
- Core



CRUST :

- The crust is the outermost solid part of the earth. It is fragile.
- The thickness of the crust varies under the oceanic and continental areas. Oceanic crust is thinner as compared to the continental crust.
- The continental crust is thicker in the areas of major mountain systems.
- The crust is made up of heavier rocks having a density of 3 g/cm3.
- The kind of rock seen in the oceanic crust is basalt.
- The mean density of material in the oceanic crust is 2.7 g/cm3.
- Silica (Si) and Aluminium (Al) are major constituent minerals. Hence it is often termed as SIAL. Also, sometimes SIAL is used to refer to the Lithosphere.

MANTLE :

- The portion of the interior beyond the crust is called the mantle.
- It is in a solid-state. It has a density higher than the crust portion.
- The thickness ranges from 10-200 km. The mantle extends from Moho's discontinuity to a depth of 2,900 km.
- The asthenosphere is the upper portion of Mantle.
- It is the chief source of magma that finds its way to the surface during volcanic eruptions.
- The crust and the uppermost part of the mantle are called the lithosphere.
- The major constituent elements of the mantle are Silicon and Magnesium and hence it is also termed as SIMA

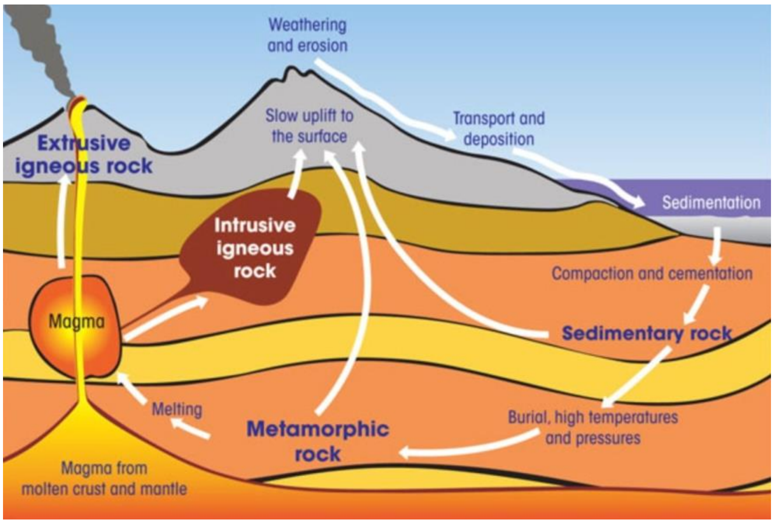
CORE :

- The core-mantle boundary is positioned at the depth of 2,900 km.
- The inner core is in the solid-state whereas the outer core is in the liquid state.
- The core is made up of very heavy material mostly constituted by nickel and iron. Hence it is also called the "nife" layer

1.3.2 SOIL GENESIS

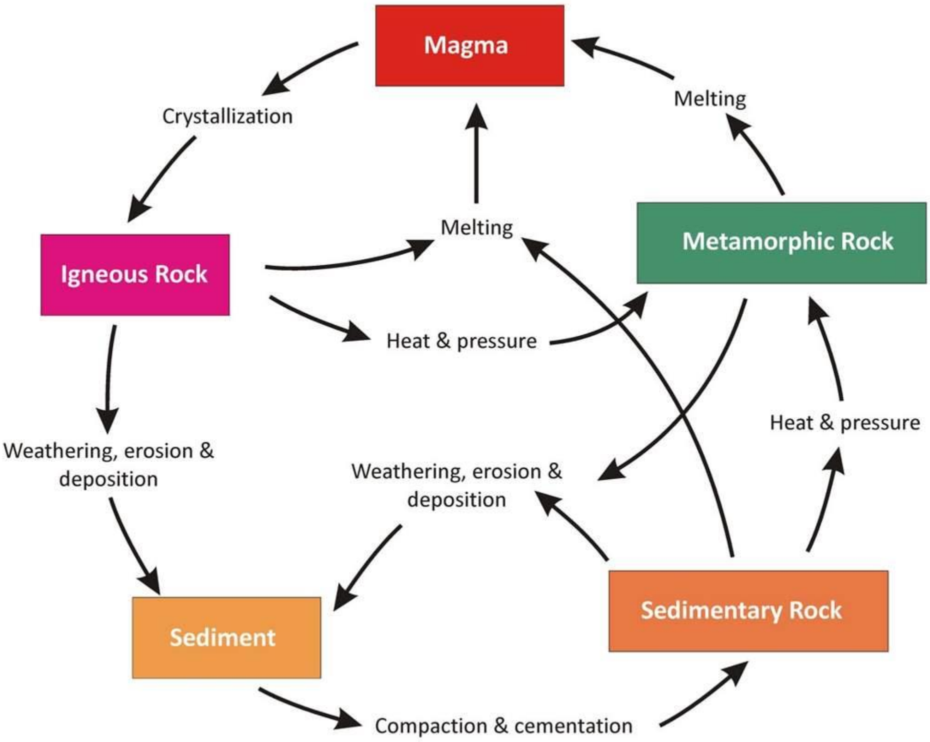
Soil genesis or pedogenesis is *the study of the origin and formation of soil*. In the Earth's surface, rocks extend up to as much as 20 km depth. The major rock types are categorized as igneous, sedimentary, and metamorphic.

Igneous rocks (from the Latin word for fire) form when hot, molten rock crystallizes and solidifies. The melt originates deep within the Earth near active plate boundaries or hot spots, then rises toward the surface.



Sedimentary rocks are formed from pre-existing rocks or pieces of once-living organisms. They form from deposits that accumulate on the Earth's surface. Sedimentary rocks often have distinctive layering or bedding.

Metamorphic rocks started out as some other type of rock, but have been substantially changed from their original igneous, sedimentary, or earlier metamorphic form. Metamorphic rocks form when rocks are subjected to high heat, high pressure, hot mineral-rich fluids or, more commonly, some combination of these factors.



1.3.3 MECHANISM OF SOIL FORMATION – WEATHERING

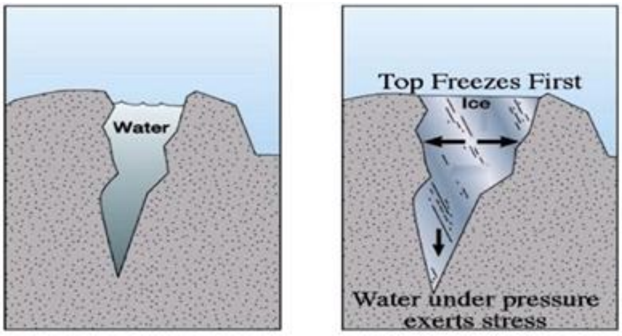
Weathering is the deterioration of rocks, soils and minerals as well as wood and artificial materials through contact with water, atmospheric gases, and biological organisms. Weathering occurs in situ (on site, with little or no movement), and so is distinct from erosion, which involves the transport of rocks and minerals by agents such as water, ice, snow, wind, waves and gravity.

The materials left over after the rock breaks down combine with organic material to create soil. Many of Earth's landforms and landscapes are the result of weathering processes combined with erosion and re-deposition. Weathering is a crucial part of the rock cycle, and sedimentary rock, formed from the weathering products of older rock, covers 66% of the Earth's continents and much of its ocean floor

1.3.3.1 TYPES OF WEATHERING

Weathering processes are divided into3 types.

1. Physical weathering
2. Chemical weathering
3. Biological weathering



Freeze and Thaw

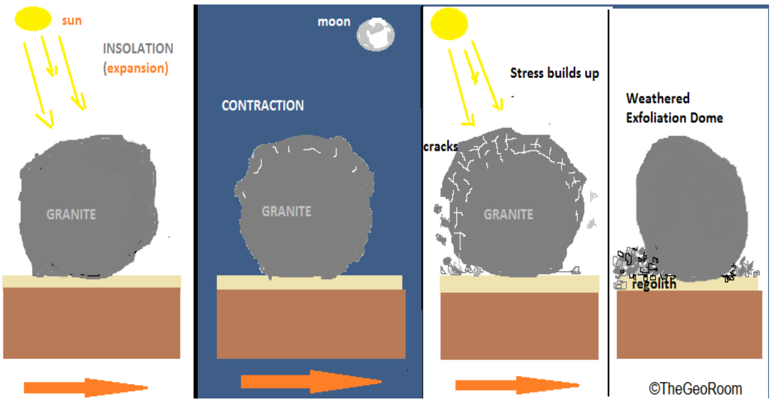
Water is the principal agent behind both physical and chemical weathering, though atmospheric oxygen and carbon dioxide and the activities of biological organisms are also important. Chemical weathering by biological action is also known as biological weathering.

PHYSICAL WEATHERING

Physical weathering involves the breakdown of rocks and soils through the mechanical effects of heat, water, ice, or other agents. Physical weathering reduces the size of the parent rock material, without any change in the original composition of the parent rock. Physical or mechanical processes taking place on the earth's surface include the actions of water, frost, temperature changes, wind and ice. They cause disintegration and the products are mainly coarse soils.

The main processes involved are exfoliation, unloading, erosion, freezing, and thawing. The principal cause is climatic change. In exfoliation, the outer shell separates from the main rock. Heavy rain and wind cause erosion of the rock surface. Adverse temperarture changes produce fragments due to different thermal coefficients of rock minerals. The effect is more for freeze-thaw cycles. There are two main types of physical weathering:

- ***Freeze –thaw*** occurs when water continually seeps into cracks, freezes and expands, eventually breaking the rock part
- ***Exfoliation*** occurs as cracks develop parallel to the land surface a consequence of the reduction in pressure during uplift and erosion



Physical weathering happens especially in places where there is little soil and few plants grow, such as in mountain regions and hot deserts. It occurs as either through repeated melting and freezing of water (mountains and tundra) or through expansion and contraction of the surface layer of rocks that are baked by the sun (hot deserts).

CHEMICAL WEATHERING

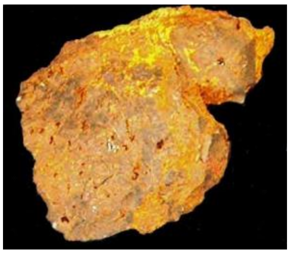
Chemical weathering involves the chemical reaction of water, atmospheric gases, and biologically produced chemicals with rocks and soils. Chemical weathering is caused by rain water reacting with the mineral grains in rocks to form new minerals (clays) and soluble salts. These reactions occur particularly when the water is slightly acidic. These chemical processes need water, and occur more rapidly at higher temperature, so warm, damp climates are best. Chemical weathering (especially hydrolysis and oxidation) is the first stage in the production of soils. Chemical weathering of rock minerals generally occurs more quickly in hot, humid climatic regions.

OXIDATION

Oxidation is the reaction of rock minerals with oxygen, thus changing the mineral composition of the rock. When minerals in rock oxidize, they become less resistant to weathering. Iron, a commonly known mineral, becomes red or rust colored when oxidized.

The iron in olivine (Fe_2SiO_4) is reduced and the iron in limonite ($Fe_2O_3.H_2O$) is oxidized. In addition, the release of silicon and hydration makes the mineral more susceptible to physical weathering.

OLIVINE



LIMONITE

REDUCTION

When oxidised minerals are positioned in a situation where oxygen is absent, the reduction occurs. Such circumstances exist commonly below the water table, waterlogged ground and in areas of stagnant water. Red colour of iron upon reduction turns to greenish or bluish-grey. These weathering processes are interconnected

CARBONATION

Carbonation is the process of rock minerals reacting with carbonic acid. Carbonic acid is formed when water combines with carbon dioxide. Carbonic acid dissolves or breaks down minerals in the rock.

$$CO_2 + H_2O \longrightarrow H_2CO_3 \qquad CaCO_3 + H_2CO_3 \longrightarrow 2Ca^+ + 2HCO_3^-$$

(carbon dioxide + water → carbonic acid) (calcite + carbonic acid → calcium + bicarbonate)

HYDROLYSIS

Hydrolysis is a chemical reaction caused by water. Water changes the chemical composition and size of minerals in rock, making them less resistant to weathering. Click on the video clip below to see hydrolysis of a relatively weathering resistant mineral, feldspar. When this mineral is completely hydrolyzed, clay minerals and quartz are produced and such elements as K, Ca, or Na are released. A hydrolysis reaction of orthoclase (alkali feldspar), a common mineral found in igneous rock, yields kaolinite, silicic acid, and potassium.

$$2KAlSi_3O_8 + 2H^+ + 9\,H_2O \longrightarrow H_4Al_2Si_2O_9 + 4\,H_4SiO_4 + 2K^+$$

(orthoclase + water → kaolinite + silicic acid + potassium)

SOLUTION

When substances are dissolved in acids or water, then the water or acid with dissolved substances is called a solution. This process includes the removal of solids in solution and depends upon the solubility of a mineral in weak acids or water. Many solids disintegrate and mix up as a suspension in water as they come in contact with water. Some of the soluble rock-forming minerals like sulphates, nitrates, and potassium, etc. are affected by this process. Hence, these minerals are simply leached out without leaving any remains in rainy climates and accumulate in dry regions. Minerals like calcium magnesium bicarbonate and calcium carbonate present in limestone are soluble in water containing carbonic acid and are transported away in the water as a solution. Carbon dioxide formed by decomposing organic matter along with soil water significantly assists in this reaction. Sodium chloride is also a rock-forming mineral and is vulnerable to this process of solution. Carbonation, oxidation and Hydration go hand in hand and accelerate the weathering process.

BIOLOGICAL WEATHERING

Biological weathering is the weakening and subsequent disintegration of rock by plants, animals and microbes. Growing plant roots can exert stress or pressure on rock. Although the process is physical, the pressure is exerted by a biological process (*i.e.*, growing roots). Biological processes can also produce chemical weathering, for example where plant roots or microorganisms produce organic acids which help to dissolve minerals.

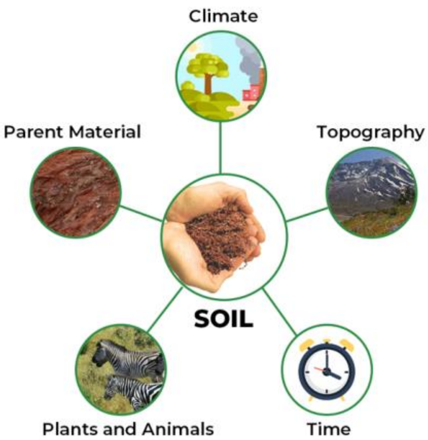
Microbial activity breaks down rock minerals by altering the rock's chemical composition, thus making it more susceptible to weathering. One example of microbial activity is lichen; lichen is fungi and algae, living together in a symbiotic relationship. Fungi release chemicals that break down rock minerals; the minerals thus released from rock are consumed by the algae. As this process continues, holes and gaps continue to develop on the rock, exposing the rock further to physical and chemical weathering.

Burrowing animals can move rock fragments to the surface, exposing the rock to more intense chemical, physical, and biological processes and so indirectly enhancing the process of rock weathering. Although physical, chemical, and biological weathering, are separate processes, some or all of the processes can act together in nature.

1.3.3.2 FACTORS AFFECTING WEATHERING

Formation of soil starts with the parent material and continues for a very long period of time taking 1000 years or more. As the parent material is weathered and / or transported, deposited and precipitated it is transformed into a soil. The parent material may be in the form of bedrock, glacial deposits, and loose deposits under water or material moving down sloping land. Following are the main factors affecting formation of soil.

1. Original or Parent Rock material
2. Climatic conditions
3. Topography
4. Organisms
5. Time



Bare rocks exposed to warm climate, frequent and heavy annual rainfalls causes faster development of soil.

ORIGINAL OR PARENT ROCK MATERIAL

It refers to the unconsolidated mineral material or organic material from which the soil is formed. Soils will carry the physical and chemical characteristics of its parent material such as color, texture, structure, mineral composition and so on. For example, if soils are formed from an area with large rocks (parent rocks) of red sandstone, the soils will also be red in color and have the same feel as its parent material. The rate of soil formation is also influenced by the parent material.

CLIMATIC CONDITIONS

It is one of the most important factors Affecting Formation of Soil. Climatic components like temperature and rainfall / precipitation are the major contributing factors influencing the effect of climate. These components affect the amount of vegetation and forest cover as well as human/animal activity in the area. The climate of an area also affects the weathering process which affects the soil formation process and speed.

TOPOGRAPHY

The shape of the land surface, its slope and position on the landscape, greatly influence the kinds of soils formed. The soil formation is also affected by surface runoff or depth to water table. Soils that developed on higher elevations and sloping areas are generally excessively drained or well drained. Steep, long slopes mean water will run down faster and potentially erode the surfaces of slopes. Permeability of the soil material; as well as the length, steepness, and configuration of the slopes, influence the kind of soil that is formed in an area.

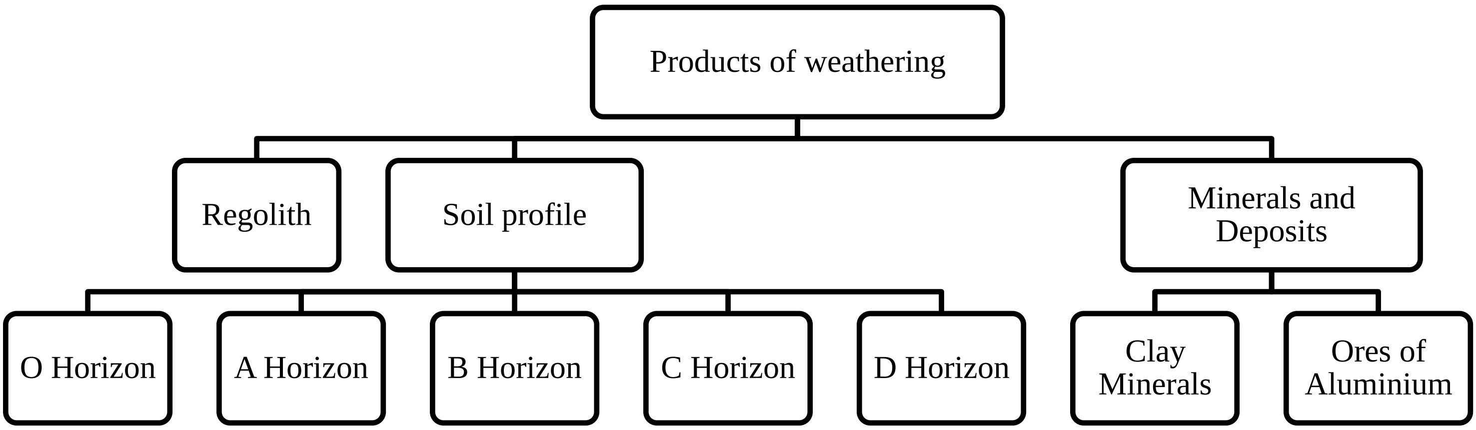
ORGANISMS

All living organisms including bacteria, fungi, vegetation, humans and animals actively influence the process of soil formation. Some types of micro-organisms promote acid conditions and change the chemistry of the soil which in turn influences the type of soil forming processes that take place. Microbial animals decompose organic materials and return the products of decomposition to the soil. Animal droppings, dead insects and animals result in additional decaying organic matter. Microorganisms also help with mineral and nutrient cycling and chemical reactions. Earthworms and burrowing animals mix the soil and change its physical characteristics. They generally make the soil more permeable to air and water. Their waste products cause aggregation of the soil particles and improve soil structure.

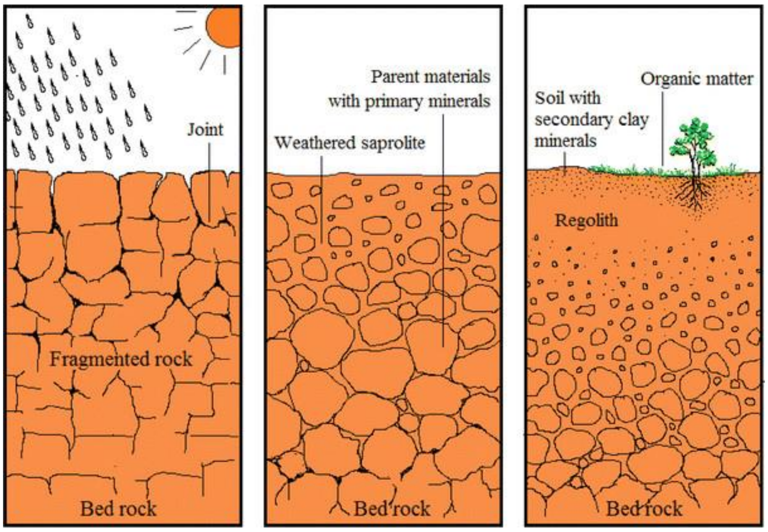
TIME

Time for all these factors to interact with the soil is also a factor. The formation of soils is a continuing process and generally takes several thousand years for significant changes to take place. These soil forming factors continue to affect soils even on “stable” landscapes. Materials are deposited on their surface, and materials are blown or washed away from the surface. Additions, removals, and alterations are slow or rapid, depending on climate, landscape position, and biological activity.

1.3.4 PRODUCTS OF WEATHERING



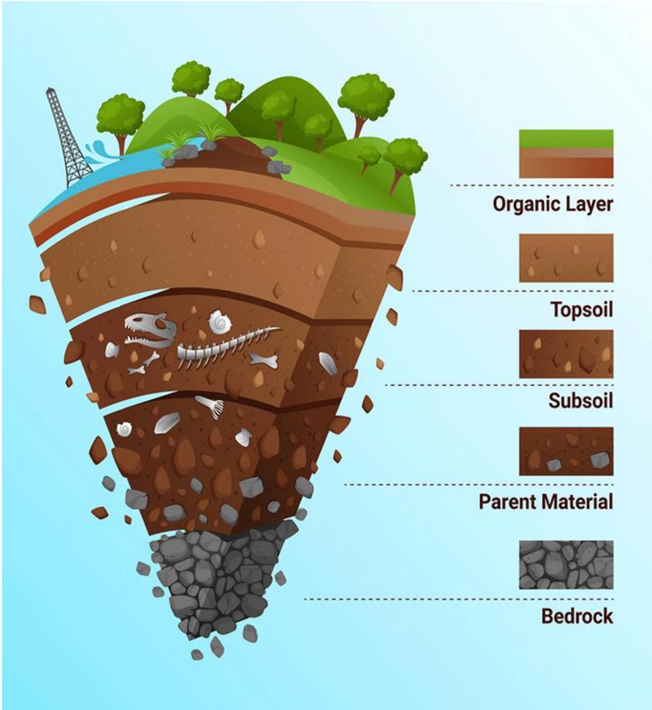
1.3.4.1 REGOLITH



Unconsolidated residual or transported material that overlies the solid rock on the earth, moon, or a planet is Regolith. Regolith, a region of loose unconsolidated rock and dust that sits atop a layer of bedrock. On Earth, regolith also includes soil, which is a biologically active medium and a key component in plant growth. Regolith serves as a source of other geologic resources, such as aluminum, iron, clays, diamonds, and rare earth elements. It also appears on the surfaces of the Moon, other planets, and asteroids; however, the material found on other celestial bodies explored so far does not contain soil. The word is the Greek term for "blanket rock."

On Earth, regolith is largely a product of weathering. Bedrock may be exposed to water or other compounds that percolate through the soil, or it may occur as an outcrop (that is, a deposit of rock exposed at Earth's surface). These chemicals can alter the rock's mineral content over time, breaking down some material into smaller components and separating it from the bedrock layer. Bedrock can also become regolith as a result of mechanical weathering, a process that breaks the rock into smaller pieces through the application of a force, such as thermal expansion, freeze-thaw cycles, or scouring by particles carried by wind and water. Plant roots can also assist the weathering process by penetrating and widening cracks already present in the rock.

1.3.4.2 SOIL PROFILE



The soil is the topmost layer of the earth's crust mainly composed of organic minerals and rock particles that support life. A soil profile is a vertical cross-section of the soil, made of layers running parallel to the surface. These layers are known as soil horizons. The soil is arranged in layers or horizons during its formation. These layers or horizons are known as the soil profile. It is the vertical section of the soil that is exposed by a soil pit. The layers of soil can easily be identified by the soil colour and size of soil particles. The different layers of soil are:

- Topsoil
- Subsoil
- Parent rock

Each layer of soil has distinct characteristics. Soil profile helps in determining the role of the soil as well. It helps one to differentiate the given sample of soil from other soil samples based on factors like its colour, texture, structure, and thickness, as well as its chemical composition.

LAYERS OF SOIL

The soil profile is composed of a series of horizons or layers of soil stacked one on top of the other. These layers or horizons are represented by letters O, A, B, C and D

O – HORIZON

The O horizon is the upper layer of the topsoil which is mainly composed of organic materials such as dried leaves, grasses, dead leaves, small rocks, twigs, surface organisms, fallen trees, and other decomposed organic matter. This horizon of soil is often black brown or dark brown in colour and this is mainly because of the presence of organic content.

A – HORIZON [TOP SOIL]

This layer is rich in organic material and is known as the humus layer. This layer consists of both organic matter and other decomposed materials. The topsoil is soft and porous to hold enough air and water. In this layer, the seed germination takes place and new roots are produced which grows into a new plant. This layer consists of microorganisms such as earthworms, fungi, bacteria, etc.

B – HORIZON [SUB SOIL]

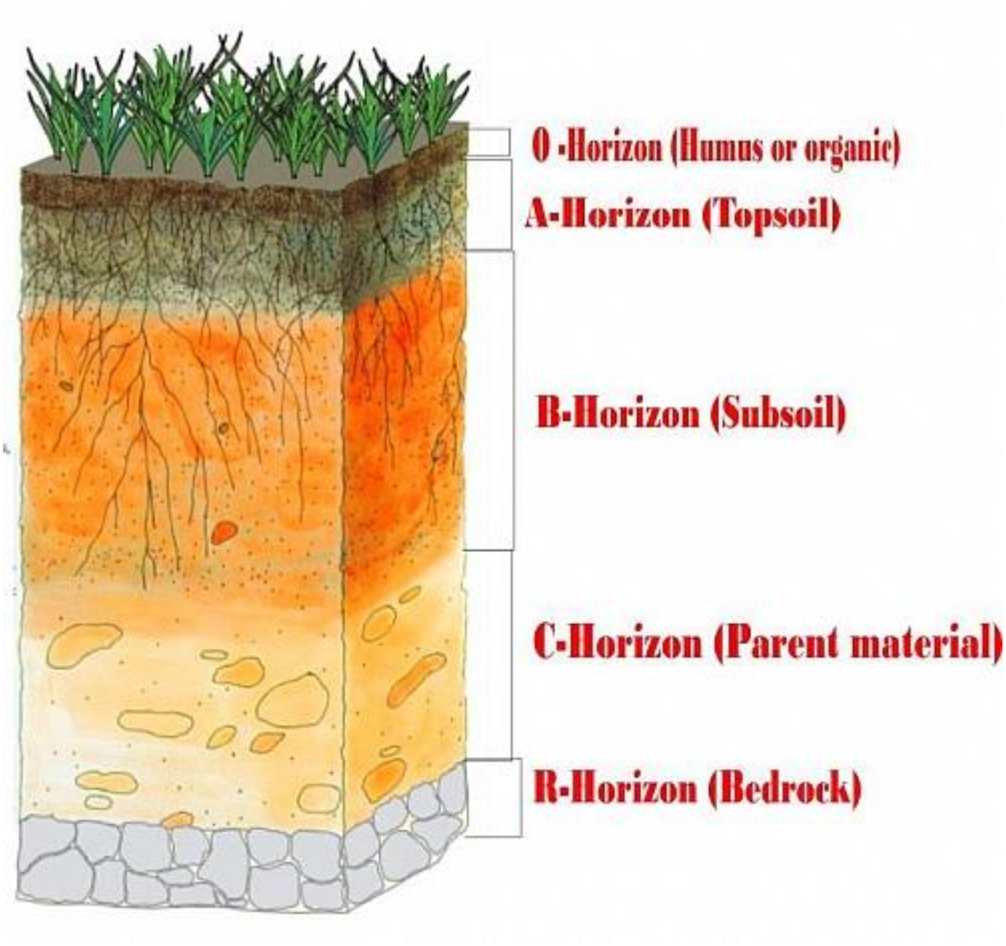
It is the subsurface horizon, present just below the topsoil and above the bedrock. It is comparatively harder and more compact than topsoil. It contains less humus, soluble minerals, and organic matter. It is a site of deposition of certain minerals and metal salts such as iron oxide. This layer holds more water than the topsoil and is lighter brown due to the presence of clay soil. The soil of horizon-A and horizon-B is often mixed while ploughing the fields.

C – HORIZON [SAPROLITE[

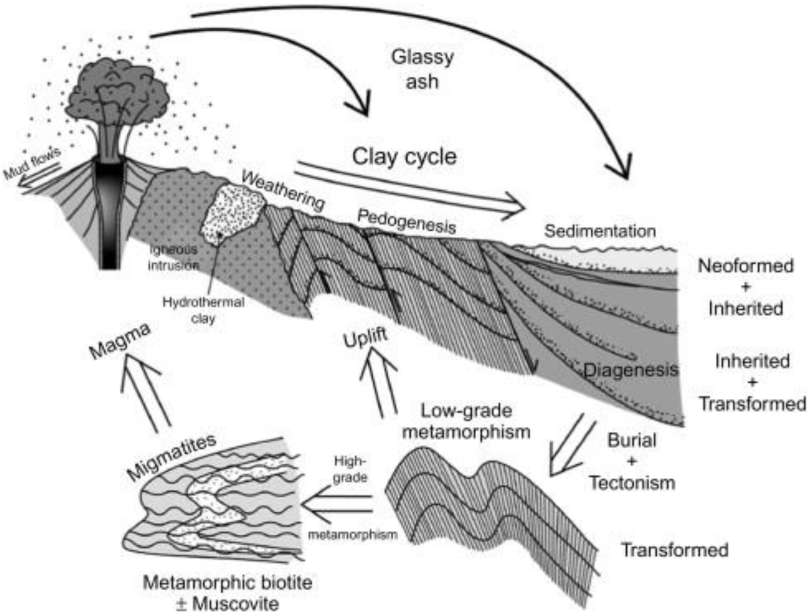
Another layer of soil that presents below the B-Horizon; this layer consists of less weathered rocks or parent rocks. It doesn't contain any organic substance in it. It is also known as saprolite.

D – HORIZON [PARENT ROCK]

D-Horizon is the layer that has partially weathered rocks or unweathered rocks. It is also known to be the foundation or the base layer of the soil. You can find manganese, granite, limestone, and basalt in this layer.

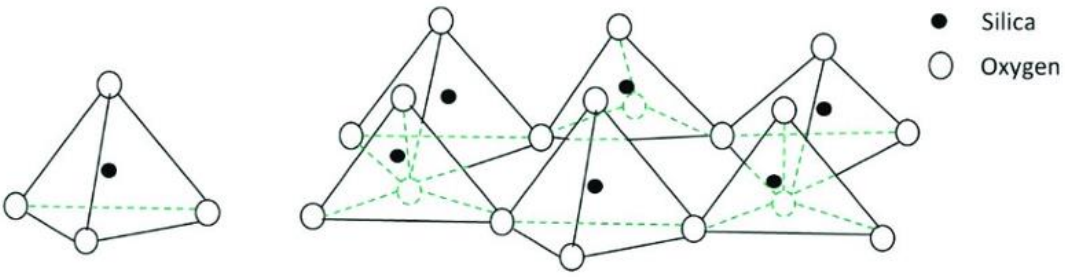


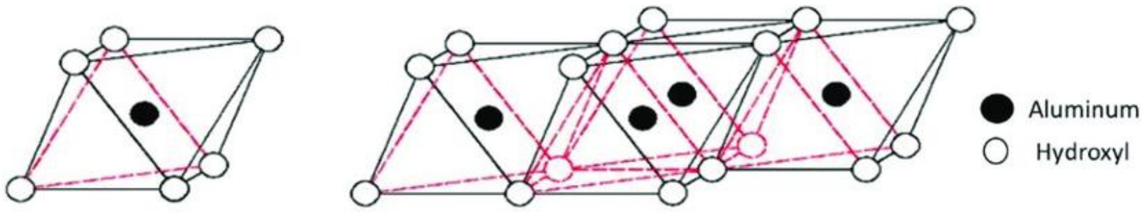
1.3.4.3 CLAY MINERALS



Clay minerals are a group of hydrous aluminosilicates that are formed from the weathering and alteration of silicate minerals. The chemical composition of clay minerals consists mainly of silica, alumina, and water. These minerals are characterized by their sheet-like structure, which is composed of layers of tetrahedrons and octahedrons.

The tetrahedral layer consists of silicon and oxygen atoms arranged in a tetrahedron shape. Each tetrahedron shares three oxygen atoms with neighboring tetrahedrons, forming a three-dimensional network. The octahedral layer consists of aluminum (or magnesium) and oxygen atoms arranged in an octahedron shape. The aluminum (or magnesium) atoms occupy the center of the octahedron, surrounded by six oxygen atoms.





The tetrahedral and octahedral layers are combined to form the basic building block of clay minerals, which is called a 2:1 layer. The 2:1 layer consists of one octahedral layer sandwiched between two tetrahedral layers. The layers are held together by weak electrostatic forces, allowing the layers to slide over one another. The layers can also absorb and exchange cations, making clay minerals important in soil chemistry.

There are several types of clay minerals, including

1. Kaolinite
2. Illite
3. Montmorillonite etc.,

Each type has a different chemical composition and structure, resulting in unique physical and chemical properties.

KAOLINITE

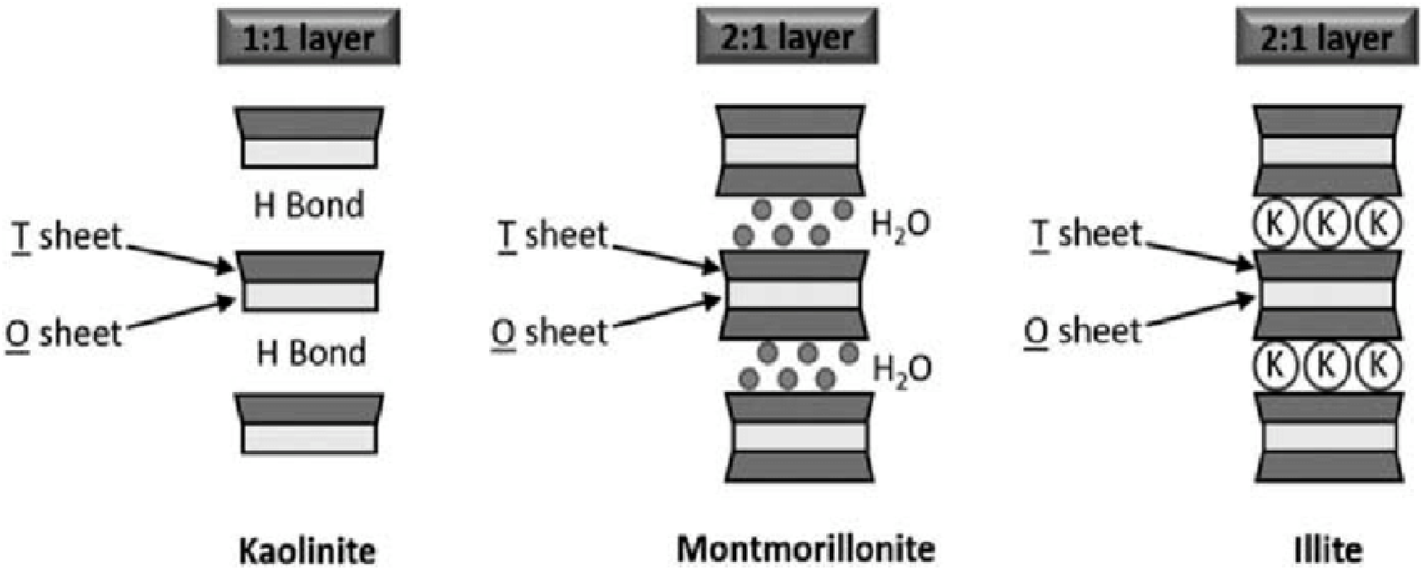
Kaolinite is a white, clay mineral that is commonly found in soils and sedimentary rocks. It has a low cation exchange capacity and a high alumina content, which makes it useful in ceramics, paper production, and as a filler in plastics and rubber.

ILLITE

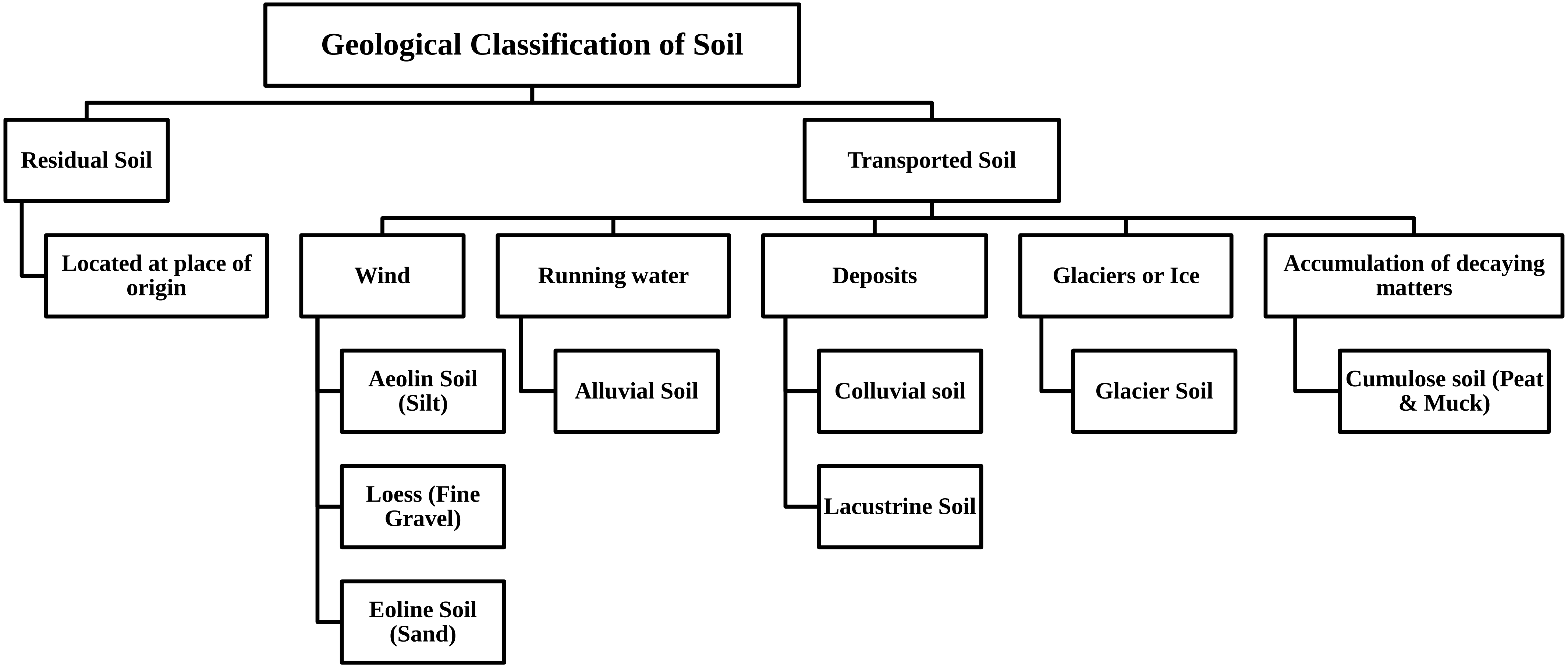
Illite is a non-swelling clay mineral that is commonly found in sedimentary rocks. It is used in the production of bricks, cement, and as a filler in paints and coatings.

MONTMORILLONITE

Montmorillonite is a smectite clay mineral that is commonly used in drilling muds, as well as in environmental remediation and as a binder in animal feed. It has a high cation exchange capacity and a high swelling capacity when hydrated.



1.3.5 GEOLOGICAL CLASSIFICATION OF SOIL



1.3.5.1 RESIDUAL SOIL

Residual soils are formed from the weathering of rocks and practically remain at the location of origin with little or no movement of individual soil particles. Weathering (due to climate effects) and leaching of water-soluble materials in the rock are the geological processes in the formation of these soils. The rate of rock decomposition is greater than the rate of erosion or transportation of weathering material and results in the accumulation of residual soil. As the leaching action decreases with depth, there is a progressively lesser degree of rock weathering from the surface downwards, resulting in reduced soil formation, until one finally encounters unaltered rock.

1.3.5.2 TRANSPORTED SOIL

Transported soils are those that have formed at one location (like residual soils) but are transported and deposited at another location.

TRANSPORTATION BY WIND

AEOLIAN SOIL is the soil deposition by high-velocity wind force. The aeolian soil has been classified as two types.

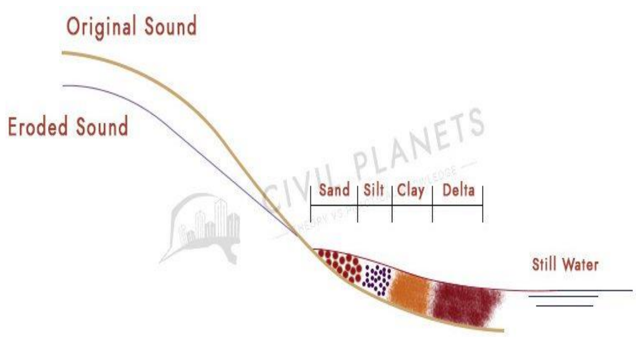
DUNE [EOLINE SOIL] – In waterless areas like desert zones, the soil is deposited in the form of dune by the substantial wind force. Dunes seem to like coarse sand having uniform shape & sizes. The bearing capacity of dune soil is low.

LOESS [LOESS SOIL] – The soil transported in very long distances is mostly silt or clay soil. Loes soil has high permeability and low density.

TRANSPORTATION BY RUNNING WATER

Major soil transportation happened with the water flow only. Both coarse and fine soil particles transported by the streaming of water. The water flows over the surface of mountains and erodes the rock. The amount of soil transportation depends on the velocity of water flow.

- When the velocity of water is high, then the large size of soil particles will be disintegrated and moved from one place to another.
- When the velocity of water slowly decreases then the coarse particles will settle down at distinct points. Remaining fine particles are deposited in the riverbank. Finally, the water gradually ended in the sea.



ALLUVIAL SOIL are deposited near the river estuary.

TRANSPORTATION BY DEPOSITS

That soil contains different size & shape of grain particles. The transportation is very less because there are no external forces to move the soil in long. The soil transported by the gravitational force is called **COLLUVIAL SOIL**.

LACUSTRINE DEPOSITS are sedimentary rock formations which formed in the bottom of ancient lakes. A common characteristic of lacustrine deposits is that a river or stream channel has carried sediment into the basin. Lacustrine deposits form in all lake types including rift graben lakes, oxbow lakes, glacial lakes, and crater lakes. Lacustrine environments, like seas, are large bodies of water. They share similar sedimentary deposits which are mainly composed of low-energy particle sizes. Lacustrine deposits are typically very well sorted with highly laminated beds of silts, clays, and occasionally carbonates. In regards to geologic time, lakes are temporary and once they no longer receive water, they dry up and leave a formation.

TRANSPORTATION BY GLACIERS / ICE

The large size of glaciers is melting due to global warming. It becomes a high-velocity flow of water which contains a different size & shape of soil particles. The soil deposits made by the glaciers melting are called **till**, and the soil deposited by the glaciers is called **glaciofluvial soil**. The glaciofluvial soil contains good shear strength, and it can be compact as required density.

ACCUMULATION OF DECAYING MATTERS

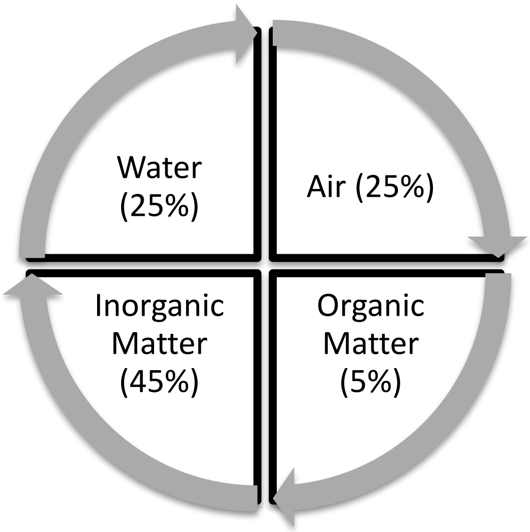
Organic (or **CUMULOSE**) deposits have developed in place from plant residues and have been preserved by a high water table (or some other factor retarding decomposition). These deposits are widespread and not restricted to any climatic zone.

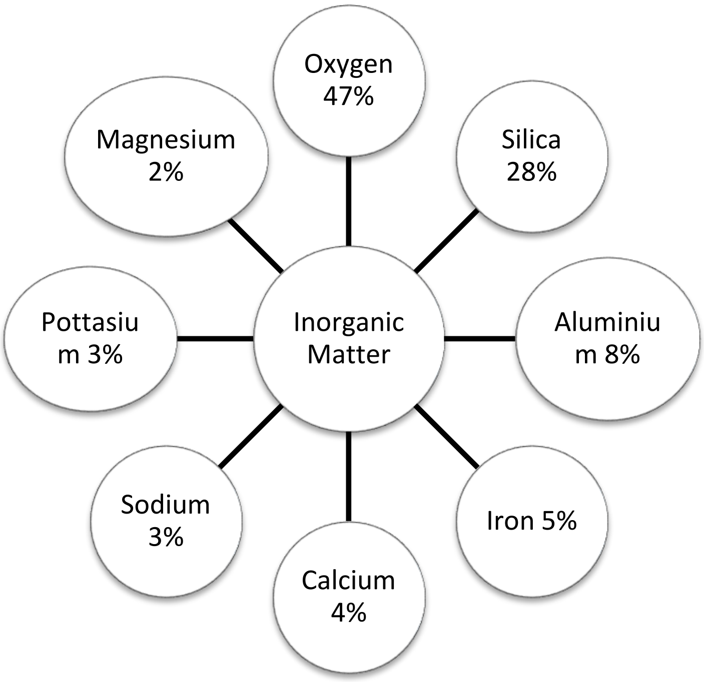
S.No	RESIDUAL SOIL	TRANSPORTED SOIL
1	These soils are found at the same location where they have been originated	Primarily, the rock materials are weathered and then they are moved from their original location to a new place by one or more erosion agents to form transported soils
2	The parent material is the original bedrock	The parent material is different from the underlying bedrock
3	The soil has the minerals similar to those in the bed rock	The soil will have a mineral composition different from that of the bedrock
4	Stress history is not important for this soil	Stress history is very important because it make changes in the primary grain packing, which leads to the settlement of consolidation

1.4 PROPERTIES OF SOIL

1.4.1 COMPOSITION OF SOIL

An estimated 70% of the earth's surface is covered with water, while the remaining 30% constitutes land. The layer of the earth that is composed of soil and is influenced by the process of soil formation is called pedosphere.





1.4.1.1 TYPES OF SOIL BASED ON ITS COMPOSITION

1. Clay Soil
2. Sandy Soil
3. Silty Soil
4. Peaty Soil
5. Chalky Soil
6. Loamy Soil

1. CLAY SOIL

- Feels lumpy and sticky when very wet
- Rock-hard when dry
- Clay drains poorly
- Few air spaces
- Warms slowly in spring
- Heavy to cultivate
- If drainage is improved, plants grow well as it holds more nutrients than many other soils

2. SANDY SOIL

- Free-draining soil
- Gritty to the touch
- Warms up quickly in spring
- Easy to cultivate
- Dries out rapidly
- May lack nutrients, which are easily washed through the soil in wet weather (often called a "hungry" soil.)

3. SILTY SOIL

- Smooth and soapy to the touch
- Well-drained soil and retains moisture
- Richer in nutrients (more fertile) than sandy soil
- Easier to cultivate than clay
- Heavier than sand
- Soil structure is weak and easily compacted
- A very good soil if well managed

4. PEATY SOIL

- Contains a much higher proportion of organic matter (peat) because the soil's acidic nature inhibits decomposition
- But this means there are few nutrients
- Dark in colour
- Warms up quickly in spring
- Highly water retentive and may require drainage if the water table is near the surface
- Fantastic for plant growth if fertiliser is added

5. CHALKY SOIL

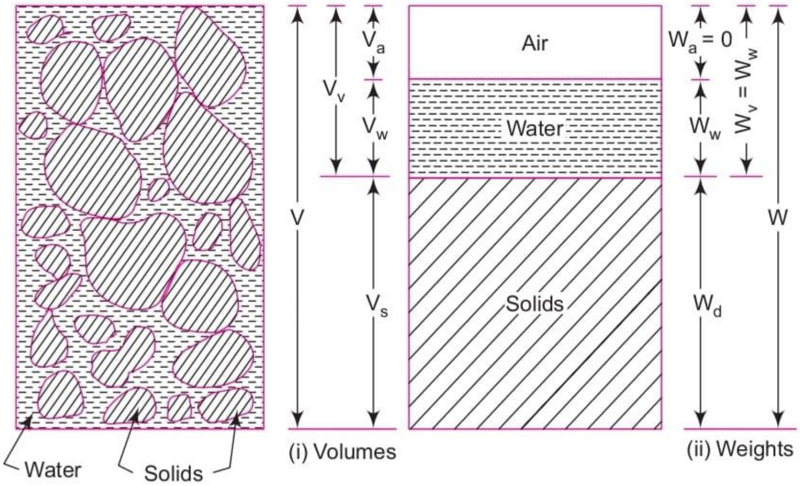
- Alkaline, with a pH of 7.5 or more
- Usually stony
- Free draining
- Often overlays chalk or limestone bedrock
- This means some minerals, such as manganese (Mg) and iron (Fe), become unavailable to plants, causing poor growth and yellowing of leaves
- This can be remedied by adding fertilisers

6. LOAMY SOIL

- The perfect soil
- Good structure
- Drains well
- Retains moisture
- Full of nutrients
- Easy to cultivate
- Warms up quickly in spring and doesn't dry out in summer
- Consider yourself very lucky if you have this soil

1.4.2 PHASE SYSTEM OF SOIL

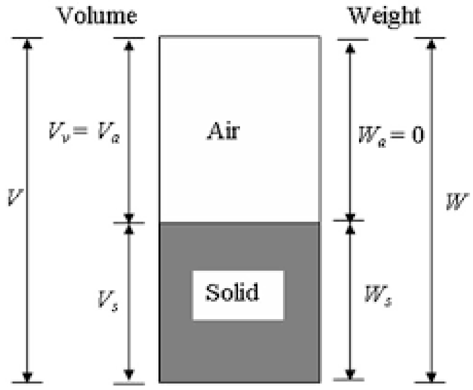
Soil deposits consist of the soil particles and the void space between the particles. Soil mass is generally referred to as three-phase system because it consists of solid particles, liquid, and gas. For many civil engineering purposes, the liquid may be considered to be water and the gas as air with the exception geo-environmental, and oil and gas applications.



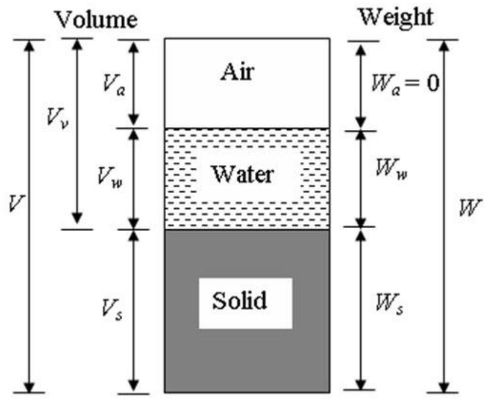
(a) Element of natural soil (b) Element separated into three phases

Phase system of soil is of three types

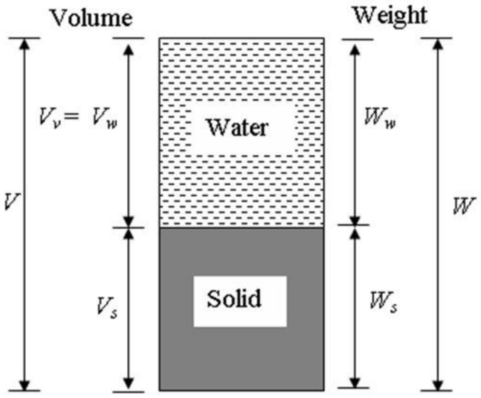
- Dry Soil
- Partially Saturated soil
- Fully Saturated Soil



DRY SOIL



PARTIALLY SATURATED SOIL



FULLY SATURATED SOIL

1.4.3 BASIC PROPERTIES OF SOIL

Physical properties of soil include colour, texture, structure, porosity, density, temperature, and air. The colours of soil vary widely from place to place and indicate some properties like organic matter, water, and redox conditions of the soil. Soil texture, structure, porosity, density, are related to the types of soil particles and their arrangement.

SOIL COLOUR: Soils are of different colours (brown, yellow, red) depending on oxidised or ferric iron compounds. The darker the colour of the soil, the more organic content it contains. The red colour of the soil is due to the presence of iron oxide and the black colour soil is rich in minerals and humus

SOIL TEXTURE: Soil texture definition (such as loam, sandy loam, or clay) refers to the proportion of sand, silt, and clay-sized particles that make up the mineral fraction of the soil. **Sand and silt** are of no importance to the soil as they don't contribute to the soil's ability to restore water and nutrients. **Clay** is an active part of soil texture as it has a small size and has a large amount of surface area per unit mass and it helps in storing water and ions. The texture of soil helps to know about the amount of water that soil can hold, the rate of water movement through the soil, how workable and fertile the soil is.

WATER ABSORPTION: Soil is able to absorb water because of its porosity. Water holding capacity is different for different types of soils. Sand absorbs less water than clay. Sandy soil water holding capacity is less than clay soil and loamy soil. Clay soil holds more water than sandy soil.

SOIL STRUCTURE: Soil structure can be defined as the way individual particles of sand, silt, and clay are assembled together. Single particles when assembled appear as larger particles. These are called aggregates. Humus is a major deciding factor to know about the structure of soil because it causes the soil to become more porous and allows water and air to penetrate deep underground. The major types of soil structure is

- Very fine or very thin
- Fine or thin
- Medium
- Coarse or thick
- Very coarse or very thick

SOIL TEMPERATURE: Soil temperature refers to the measurement of the ground's inherent warmth. It controls the chemistry and biology of the ground and the atmospheric-ground gas exchange. You may also encounter the term "soil surface temperature," which is the measurement of warmth/coldness in the top 4 inches (10 cm) of the ground.

WATER CONTENT: The amount of water or moisture present in the soil is the water content. It is the ratio of weight of water to weight of soilds.

$$w = \frac{W_w}{W_s} \times 100$$

DENSITY OF SOIL : It is the ratio of mass of solids per unit volume of the soil

$$\rho = \frac{Mass}{Volume}$$

UNIT WEIGHT OF SOIL : It is the ratio of weight of solids per unit volume of the soil

$$\gamma = \frac{Weight}{Volume}$$

SPECIFIC GRAVITY : It is the ratio of weight of given volume of soil solids at a given temperature to the weight of an equal volume of distilled water at that temperature

$$G = \frac{\gamma_s}{\gamma_w}$$

VOIDS RATIO : It is the Ratio of volume of voids to the volume of soil solids in the given soil mass

$$e = \frac{V_V}{V_S} = \frac{n}{1-n}$$

POROSITY : It is the Ratio of volume of voids to the total volume of given soil mass

$$n = \frac{V_V}{V} = \frac{e}{1+e}$$

DEGREE OF SATURATION : It is the ratio of volume of water present in a given soil mass to the total volume of voids in it

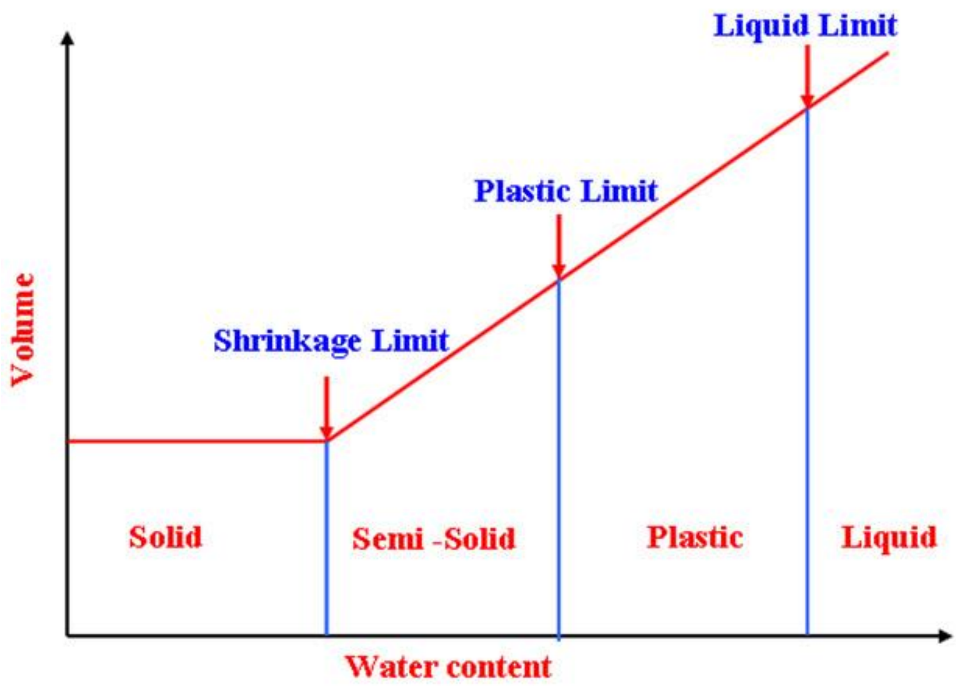
$$S = \frac{V_w}{V_V}$$

1.4.4 INDEX PROPERTIES OF SOIL

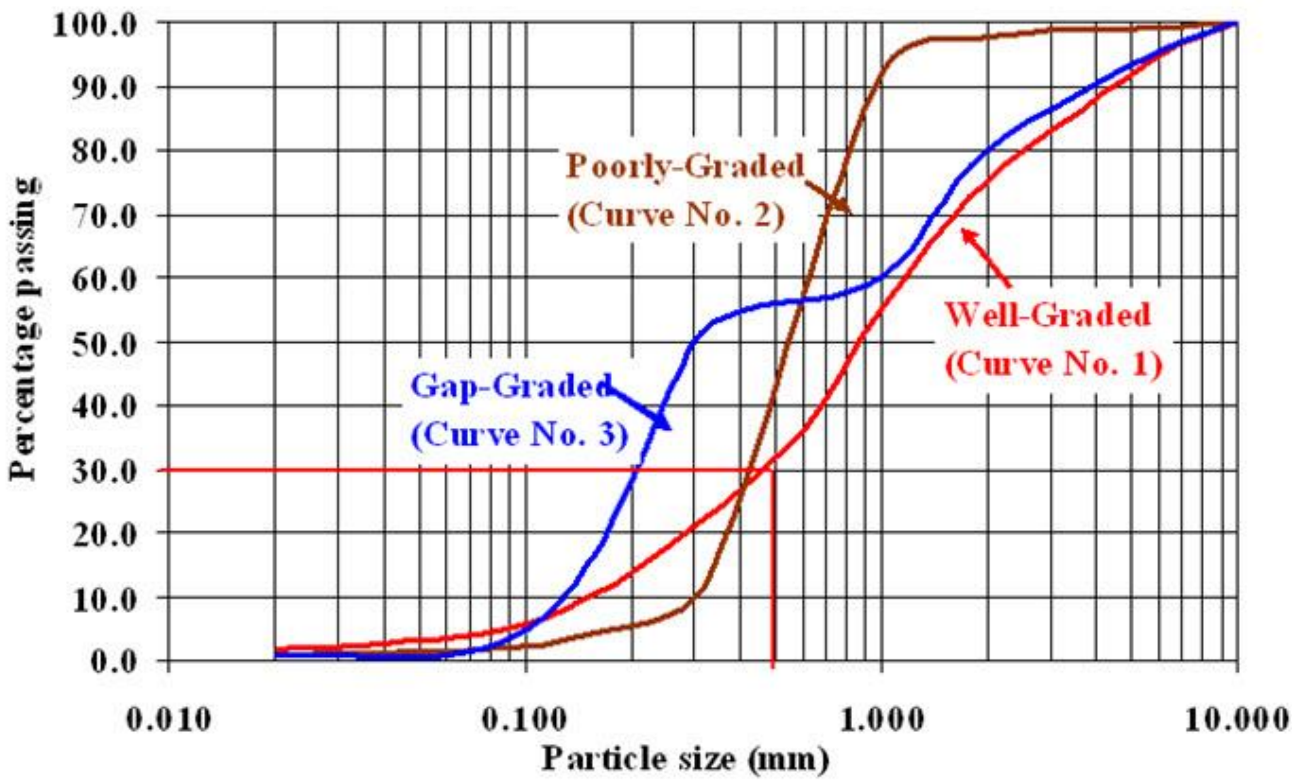
- Water content
- Specific gravity
- Particle size distribution
- Consistency limits / Atterberg Limit
- In-situ density
- Density index

CONSISTENCY LIMIT: Depending on its water content, soil may appear in one of four states: solid, semi-solid, plastic and liquid. In each state, the consistency and behavior of soil are different, and consequently so are its engineering properties. Thus, the boundary between each state can be defined based on a change in the soil's behavior. The Atterberg limits can be used to distinguish between silt and clay and to distinguish between different types of silts and clays. The water content at which soil changes from one state to the other is known as **consistency limits, or Atterberg's limit.** Various types of Atterberg limits are listed below,

- Plastic limit
- Liquid limit
- Shrinkage limit



PARTICLE SIZE DISTRIBUTION: Grain size distribution gives an idea about the percentage of various soil grains present in a dry soil sample. Grain size analysis can be carried out by sieve analysis incase of coarse-grained soil (such as sands, gravels) and hydrometer analysis incase of fine-grained soil (such as silt, clay). Incase of hydrometer analysis, Stokes's law is used to determine the grain size distribution of the soil.. In each state, the consistency and behavior of soil are different,



1.4.5 ENGINEERING CLASSIFICATION OF SOIL

In general, soils may be classified as coarse-grained (cohesionless) and fine-grained (cohesive) soil. The both coarse-grained and fine-grained soils can be further sub-divided based on their grain size. According to Indian Standard Soil Classification System (ISSCS), soil with particle size > 300 mm is called **Boulder**. Soil with particle size in between 300 mm to 80 mm is called **Cobble.** The soil is called **Gravel** if particle size is in between 80 mm to 4.75 mm. The gravel is sub-divided as **Coarse Gravel** (80 mm to 20 mm) and **Fine Gravel** (20 mm to 4.75mm). Soil with particle size in between 4.75 mm to 0.075 mm is called **Sand**. The sand is sub-divided as **Coarse Sand** (4.75 mm to 2.0 mm), **Medium Sand** (2.0 mm to 0.425 mm) and **Fine Sand** (0.425 mm to 0.075 mm). Soil with particle size less than 0.075 mm is called **Fine-Grained** soil **(Silt or Clay)** and soil with particle size in between 80 mm to 0.075 mm is called **Coarse-Grained** soil. However, soil with particle size less than 0.002 mm is called **Clay** and soil with particle size in between 0.075mm to 0.002 mm is called **Silt**.

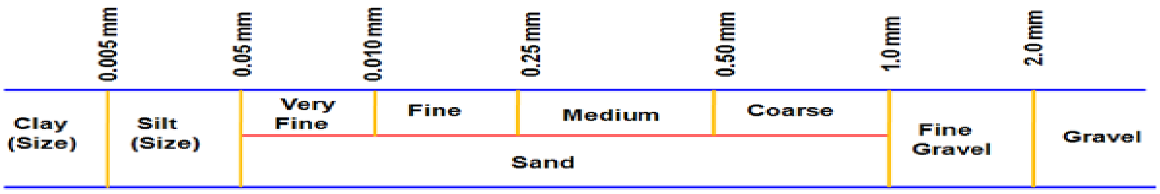
The following are the different classifications of soil,

1. Particle size classification
2. Textural classification
3. Highway research board classification
4. Unified soil classification
5. IS classification system

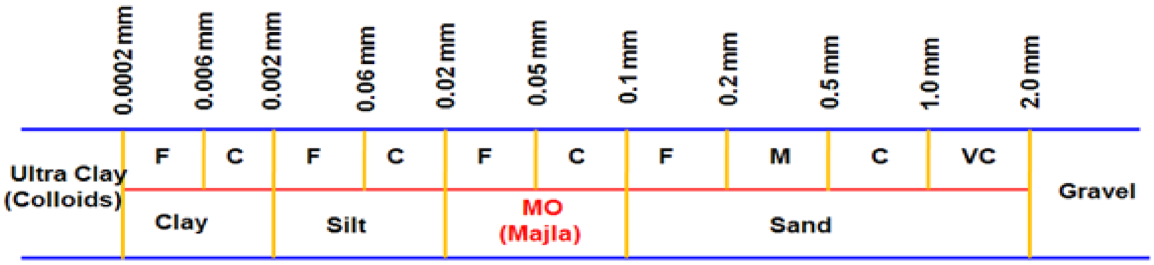
1.4.5.1 PARTICLE SIZE CLASSIFICATION OF SOIL

In this system, soils are classified according to their grain size. Terms such as gravel, sand, silt, and clay are used to indicate grain sizes. The following systems are commonly used to classify soil according to their grain size.

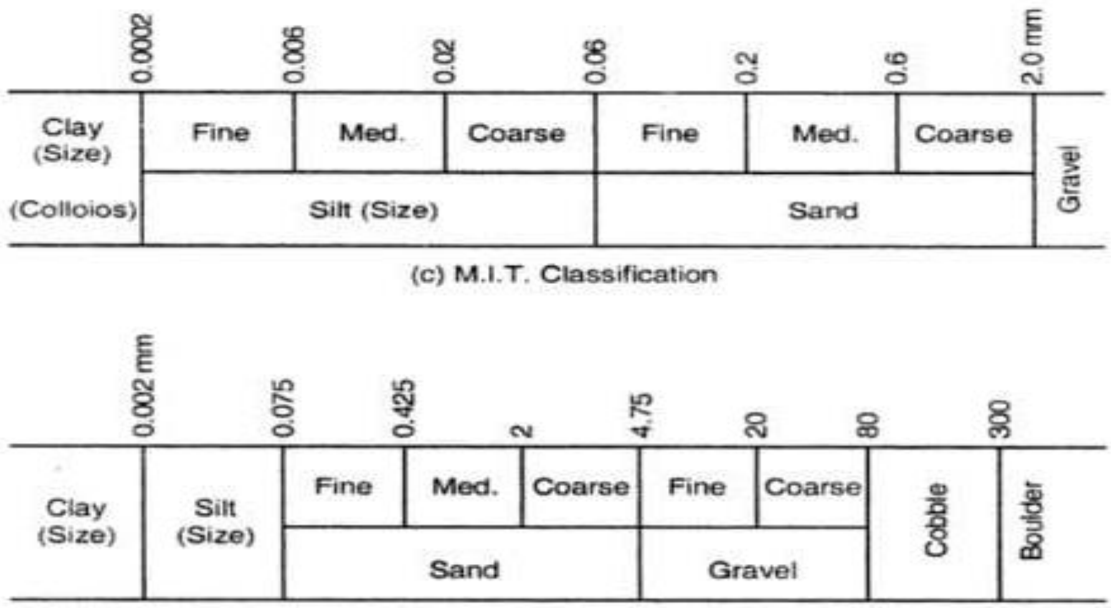
- U.S Bureau of soil and Public Road Administration (PRA) System
- International Soil Classification
- MIT Classification
- IS Classification



U.S BUREAU OF SOILS AND PRA CLASSIFICATION



INTERNATIONAL SOIL CLASSIFICATION



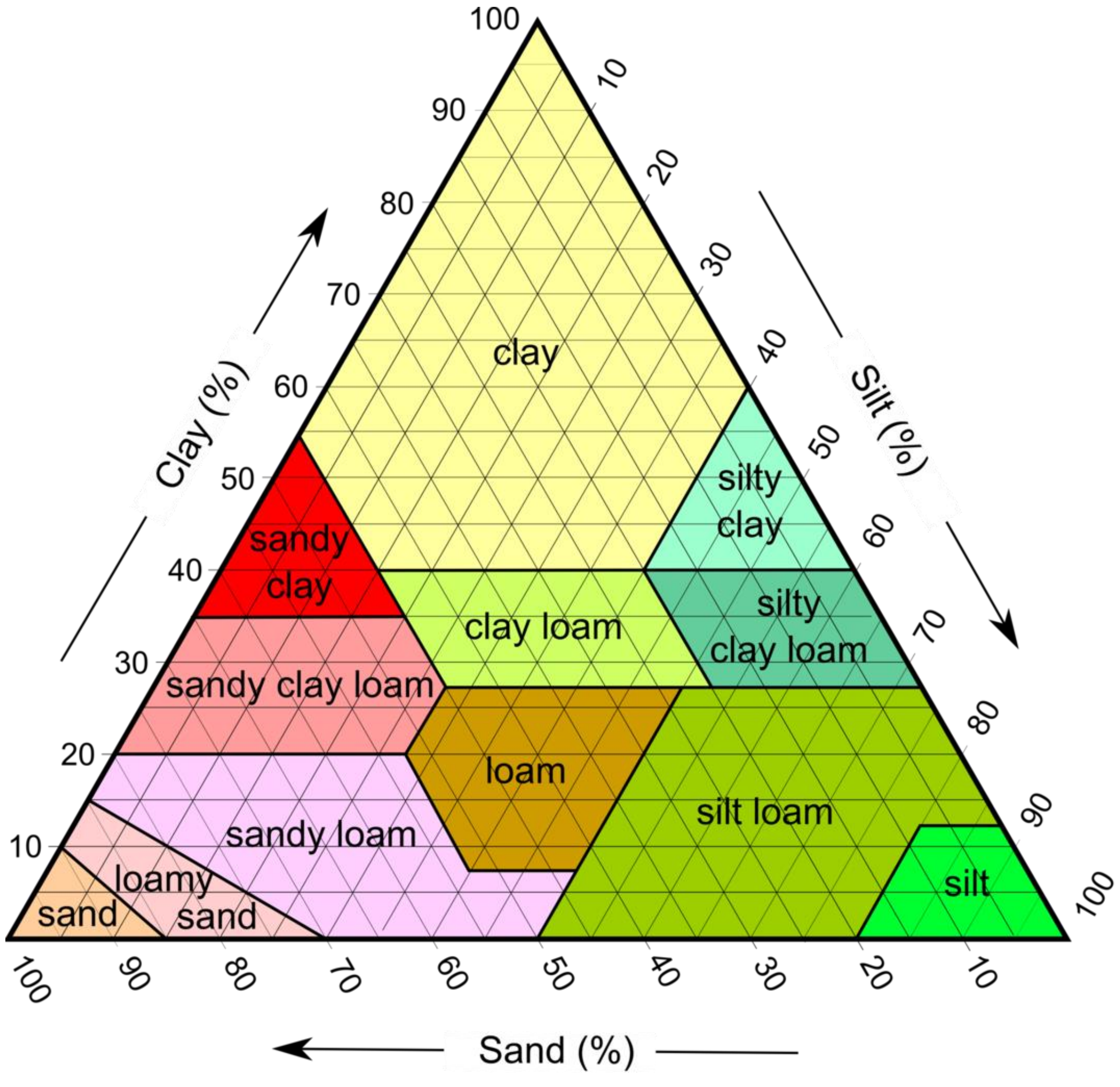
(c) M.I.T. Classification

(d) I.S. Classification (IS : 1498-1970)

GRAIN-SIZE CLASSIFICATION SCALES.

1.4.5.2 TEXTURAL CLASSIFICATION OF SOIL

The relative proportion of sand, silt and clay is called soil texture, since it is concerned with the size of mineral particles. The primary particles of sand, silt, and clay make up the inorganic solid phase of the soil. These particles often become aggregated together with each other and other parts of the soil, most importantly soil organic matter. The size of particles in mineral soil is not subject to ready change by cultural practices. Thus it is considered as a permanent feature and a basic property of soil. Mechanical analysis of soil separates i.e. the percentage of sand, silt and clay is done by Hydrometric method.



1.4.5.3 HRB CLASSIFICATION OF SOIL

The HRB soil classification system is generally adopted in highway engineering for the classification of subgrade soils. Soils are divided into seven groups A-1 to A-7. A-1, A-2 and A-3 soils are granular soils, percentage fines passing 0.075 mm sieve being less than 35. Based on particle size composition and plasticity characteristics

HRB-CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES

General Description	*Granular materials (35% or less passing 75 micron IS sieve)*							*Silt clay materials (more than 35% passing 75 micron IS sieve)*			
Group Classification	A–1		A–3	A–2				A–4	A–5	A–6	A–7
	A–1–a	A–1–b		A–2–4	A–2–5	A–2–6	A–2–7				A–7–5 A–7–6
Sieve analysis, percent passing 2.0 mim IS sieve 425 micron sieve 75 micron sieve	50 max 30 max 15 max	 50 max 25 max	 51 min 10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 425 micron sieve Liquid Limit Plasticity Index	 6 max		 NP	40 max 10 max	41 min 10 max	40 max 11 min	41 min 11 max	40 max 10 max	41 min 10 max	40 max 11 min	41 min 11 min
Group Index	Zero					4 max		8 max	12 max	16 max	20 max
Usual type of significant constituent materials	Stone fragments gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good					Fair to poor					

1.4.5.4 UNIFIED SOIL CLASSIFICATION SYSTEM

The USCS classifies soils into two broad categories: coarse-grained soils and fine-grained soils. Coarse-grained soils have a high proportion of larger particles such as sand and gravel, while fine-grained soils have a high proportion of smaller particles such as clay and silt.

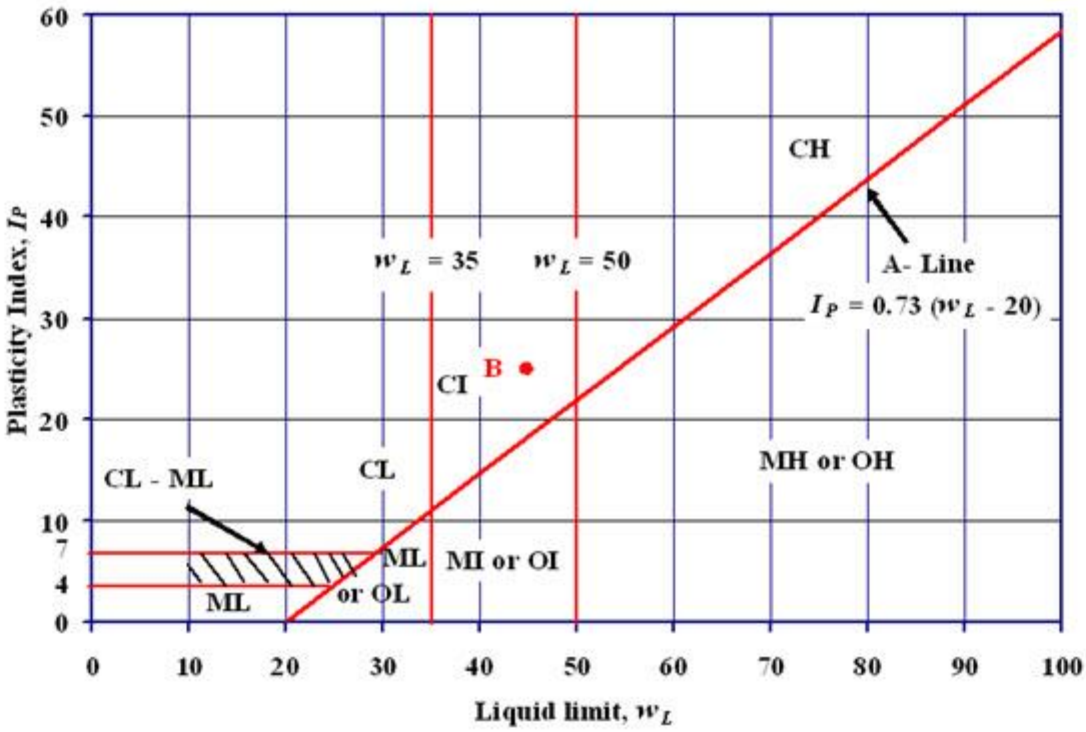
Table 4.2 Unified Soil Classification System

Major division			*Group symbol*	*Typical name*	*Classification criteria*	
Coarse-grained soils (More than 50% retained on No. 200 ASTM sieve)	Gravels 50% or more of coarse fraction retained on No. 4 ASTM sieve	Clean gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines.	Classification on the basis of percentage of fines. Less than 5% passing No. 200 ASTM sieve—GW, GP, SW, SP. More than 12% passing No. 200 ASTM sieve— GM, GC, SM, SC. 5% to 12% passing No. 200 ASTM sieve—Border-line classification requiring use of dual symbols.	$U = D_{60}/D_{10}$ greater than 4 $C_c = D^2_{30}/(D_{60} \times D_{20})$ between 1 and 3.
			GP	Poorly-graded gravels and gravel-sand mixtures, little or no fines.		Not meeting both criteria for GW.
		Gravels with fines	GM	Silty gravels, gravel-sand-silt mixtures.		Atterberg limits plot below A-line or plasticity index less than 4.
			GC	Clayey gravels, gravel-sand-clay mixtures.		Atterberg limits plot above A-line or plasticity index less than 4.
	Sands More than 50% of coarse fraction passes No. 4 ASTM sieve	Clean sands	SW	Well-graded sands and gravelly sands, little or no fines.		U greater than 6 C_c between 1 and 3.
			SP	Poorly-graded sands and gravelly sands, little or no fines.		Not meeting both criteria for SW.
		Sands with fines	SM	Silty sands, and-silt mixtures.		Atterberg limits plot below A-line or plasticity index less than 4.
			SC	Clayey sands, sand-clay mixtures.		Atterberg limits plot above A-line or plasticity index greater than 7.
Fine-grained soils (50% or more passes No. 200 ASTM Sieve)	Silts and Clays (Liquid limit 50% or less)		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.	Check Plasticity Chart	
			CL	Inorganic clays or low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.		
			OL	Organic silts and organic silty clays of low plasticity.		
	Silts and clays (Liquid limit greater than 50%)		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts.		
			CH	Inorganic clays of high plasticity, fat clays.		
			OH	Organic clays of medium to high plasticity.		
Highly organic clays			P_t	Peat, muck and other highly organic soils.	Fibrous organic matter, will char, burn, or glow. Readily identified by colour, odour, spongy feel, and fibrous texture.	

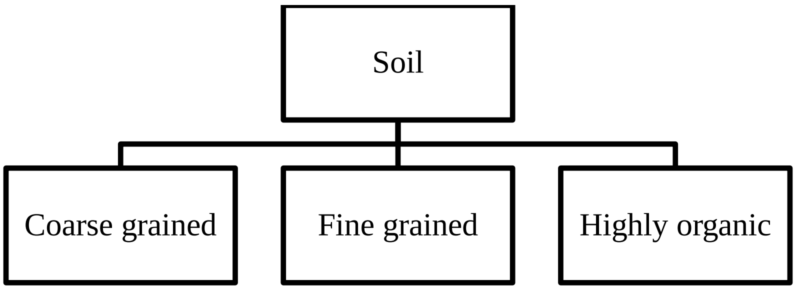
Note: Boundary classification: Soils possessing characteristics of two groups are designated by combinations of group symbols — for example, GW-GC, well-graded, gravel-sand mixture with clay binder

1.4.5.5 INDIAN STANDARD CLASSIFICATION SYSTEM

According to this system, the symbols of the various soils are as: Gravel (G), Sand (S), Silt or Silty (M), Clay or Clayey (C), Organic (O), Peat (Pt), Well graded (W), Poorly graded (P). To classify the fine-grained soil, plasticity chart is used. The difference between the plasticity charts used for Unified Soil Classification System (USCS) and Indian Standard Soil Classification System (ISSCS) is that in USCS, the soil is classified as High Plasticity (if liquid limit >50%) or Low Plasticity (if liquid limit < 50%) soil, but in ISSCS, the soil is classified as High Plasticity (if liquid limit >50%) or Intermediate Plasticity (if liquid limit is in between 35% to 50%) or Low Plasticity (if liquid limit < 35%). For example, if a soil sample has liquid limit (wL) 45% and plasticity index (IP) 25, according to the Unified Soil Classification System (USCS) the point is above 'A' line (point A in Figure 4.1) and it is classified as CL. However, according to Indian Standard Soil Classification System (ISSCS) the point is also above 'A' line, but it is classified as CI.

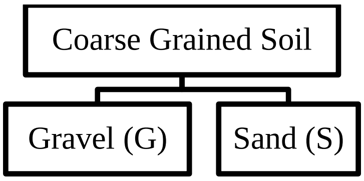


The soil is classified into 3 main categories



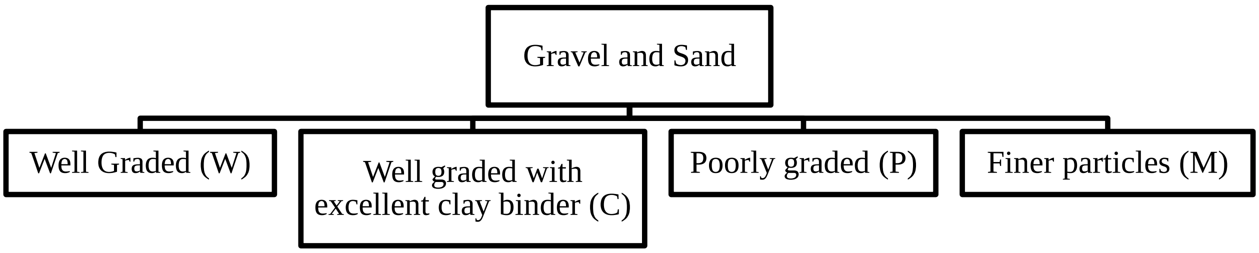
COARSE GRAINED SOIL:

- More than half of material by mass passes > 75 microns sieve size



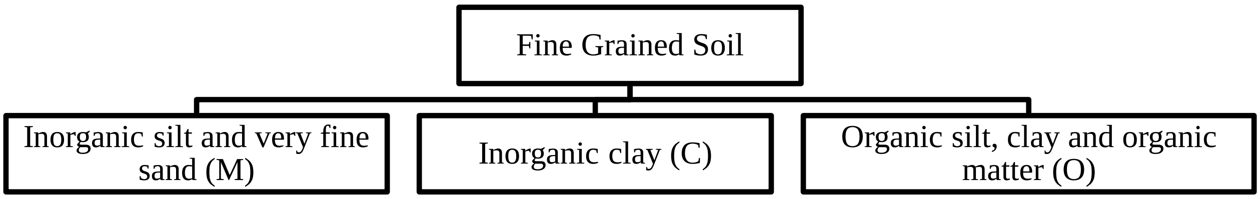
Gravel (G): More than half of the coarse fraction (75 Microns) is larger than 4.75 mm IS size

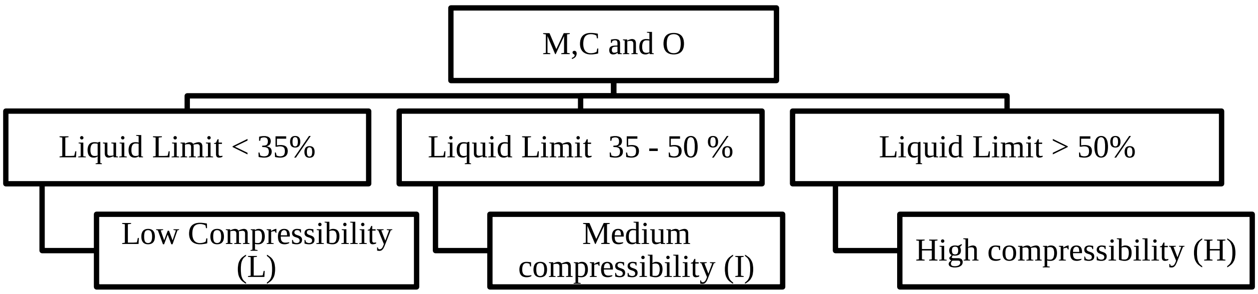
Sand (S): More than half of the coarse fraction (75 Microns) is smaller than 4.75 mm IS size



FINE GRAINED SOIL:

- More than half of material by mass passes < 75 microns sieve size





HIGHLY ORGANIC SOIL:

- Contains larger percentage of fibrous organic matter – Peat, Decomposed vegetation

Major Division	Sub Division		Sub Classification	Symbol	Classification criteria	
Coarse Grained Soil > 75µ	Gravel (G)	Clean Gravel	Well graded gravel	GW	$C_U > 4$, $C_C = 1$ to 3	Passing 75 µ sieve size < 5% - GW,GP,SW,SP > 12% - GM,GC,SM,SC 5-12% - Border line case
			Poorly graded Gravel	GP	-	
		Gravel with fines	Clayey Gravel	GC	Plasticity Chart, Above A line $I_P > 7$	
			Silty gravel	GM	Plasticity Chart, Above A line $I_P < 4$	
	Sand (S)	Clean Sand	Well graded sand	SW	$C_U > 6$, $C_C = 1$ to 3	
			Poorly graded sand	SP	-	
		Sand with fines	Clayey sand	SC	Plasticity Chart, Below A line $I_P > 7$	
			Silty sand	SM	Plasticity Chart, Below A line $I_P < 4$	
Fine Grained soil < 75µ	Silt (M)	High Compressible silt		MH	Plasticity Chart $W_L > 50\%$	$C_U = \frac{D_{60}}{D_{10}}$ $C_U = \frac{(D_{30})^2}{D_{10}\ X\ D_{60}}$
		Intermediate Compressible silt		MI	Plasticity Chart $W_L = 35\text{-}50\%$	
		Low Compressible silt		ML	Plasticity Chart $W_L < 35\%$	
	Clay (C)	High Compressible clay		CH	Plasticity Chart $W_L > 50\%$	
		Intermediate Compressible clay		CI	Plasticity Chart $W_L = 35\text{-}50\%$	
		Low Compressible clay		CL	Plasticity Chart $W_L < 35\%$	
Organic Soil	High Compressible organic clay			OH	Plasticity Chart $W_L > 50\%$	
	Intermediate Compressible organic clay and silt			OI	Plasticity Chart $W_L = 35\text{-}50\%$	
	Low Compressible organic silt			OL	Plasticity Chart $W_L < 35\%$	
	Highly organic soil			P_t	Peat	

1.5 BEARING CAPACITY OF SOIL

The bearing capacity of soil is defined as the capacity of the soil to bear the loads coming from the foundation. The pressure which the soil can easily withstand against load is called allowable bearing pressure.

1.5.1 TYPES OF BEARING CAPACITY OF SOIL

ULTIMATE BEARING CAPACITY OF SOIL [q_u]

The gross pressure at the base of the foundation at which soil fails is called ultimate bearing capacity.

NET ULTIMATE BEARING CAPACITY OF SOIL [q_{nu}]

By neglecting the overburden pressure from ultimate bearing capacity we will get net ultimate bearing capacity. (Where γ = Unit weight of soil & D_f = Depth of foundation)

$$q_{nu} = q_u - \gamma D_f$$

NET SAFE BEARING CAPACITY OF SOIL [q_{ns}]

By considering only shear failure, net ultimate bearing capacity is divided by certain factor of safety will give the net safe bearing capacity. (Where F = Factor of safety)

$$q_{ns} = \frac{q_{uu}}{F}$$

GROSS SAFE BEARING CAPACITY OF SOIL [q_s]

When ultimate bearing capacity is divided by factor of safety it will give gross safe bearing capacity.

$$q_s = \frac{q_u}{F}$$

NET SAFE SETTLEMENT PRESSURE[q_{np}]

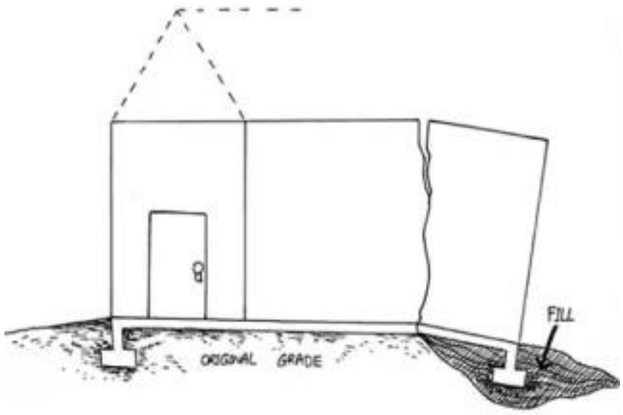
The pressure with which the soil can carry without exceeding the allowable settlement is called net safe settlement pressure.

NET ALLOWABLE BEARING PRESSURE[q_{na}]

This is the pressure we can used for the design of foundations. This is equal to net safe bearing pressure if $q_{np} > q_{ns.}$ In the reverse case it is equal to net safe settlement pressure.

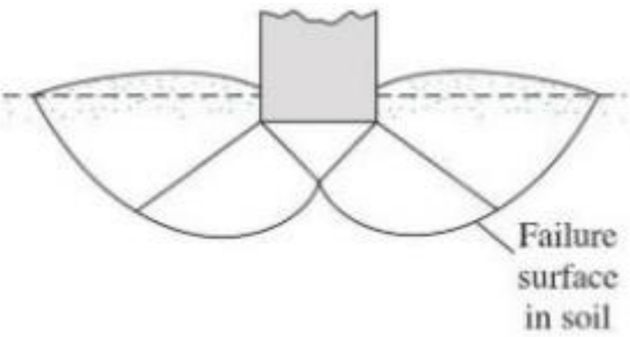
1.5 BEARING CAPACITY FAILURES

Experimental investigations shows that when a footing fails due to insufficient bearing capacity, distinct failure patterns are developed, depending upon the type of failure mechanism. Failure is accompanied by appearance of failure surfaces aand by building of sheared mass of soil. Vesic (1963) observed 3 types of bearing capacity failures



- General shear failure
- Local shear failure
- Punching shear failure

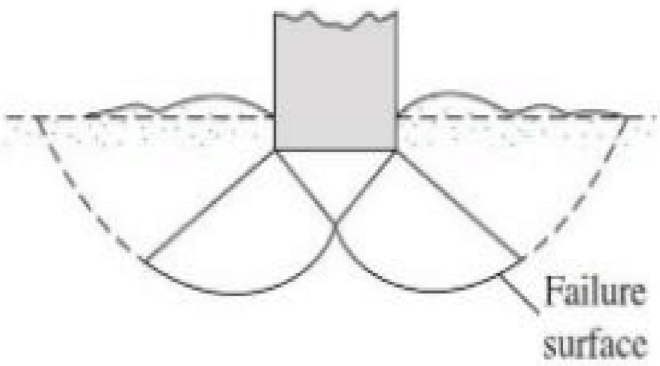
GENERAL SHEAR FAILURE :



- It has well defined failure surface, reaching upto ground surface
- There is considerable bulging of sheared mass of soil adjacent to the footing
- Failure is accompained by tilting of the footing
- Failure is sudden pronounced peak resistance
- The ultimate bearing capacity is well defined

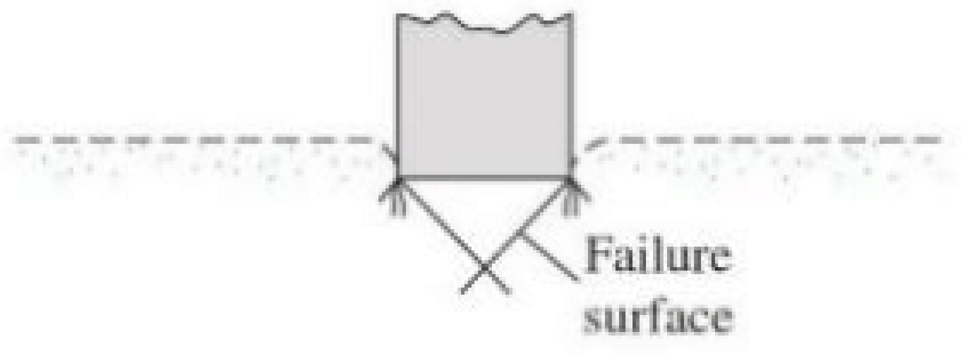
LOCAL SHEAR FAILURE :

- Failure pattern is clearly defined only immediately below the footing
- The failure surfaces do not reach the ground surface
- There is only slight bulging of soil around the footing
- Failure is not sudden and there is no tilting of footing
- Failure is defined by large settlements
- Ultimate bearing capacity is not well defined



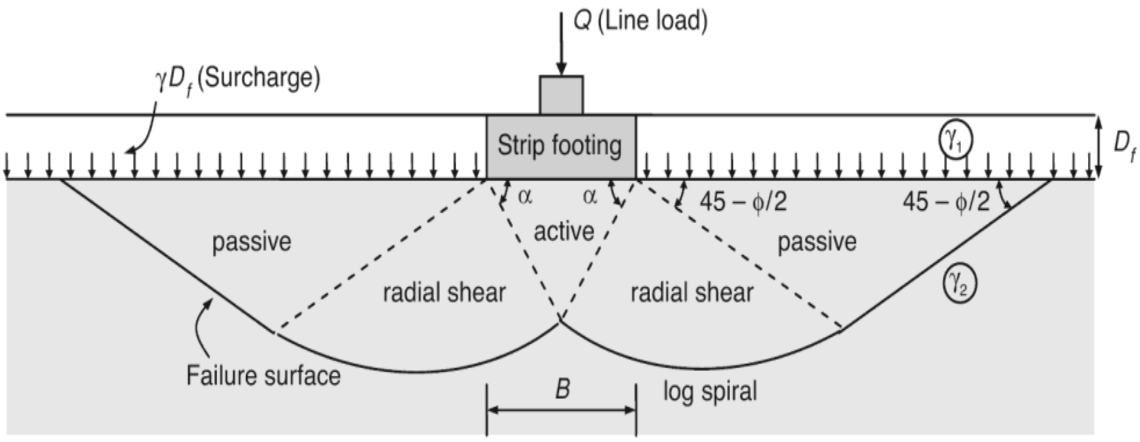
PUNCHING SHEAR FAILURE :

- No failure pattern is observed
- The failure surface, which is vertical or slightly inclined, follows the perimeter of the base
- There is no bulging of soil around the footing
- There is no tilting of footing
- Failure is characterised in terms of very large settlements
- The ultimate bearing capacity is not well defined



1.6 TERZAGHI'S ANALYSIS

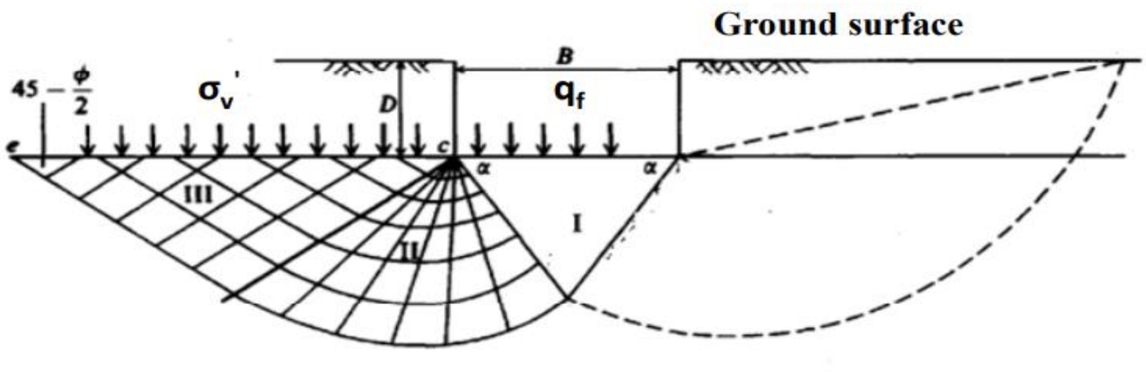
Terzaghi in 1943 gave a general bearing capacity theory for a strip foundation, called "**Terzaghi's Theory on Bearing Capacity Analysis**". For the first time, he developed his theory by incorporating the weight of the failure wedge in the analysis. Terzaghi considered a continuous footing of width B placed at a depth of D below the ground surface.



ASSUMPTIONS:

1. The soil mass is homogeneous and isotropic.
2. The shear strength of soil can be represented by Coulomb's equation.
3. The problem is two dimensional
4. The footing has rough base.
5. The ground surface is horizontal.
6. The loading is vertical and symmetrical
7. Failures zones are assumed to be formed fully.
8. The principle of superposition is applicable.
9. The failure zones do not extend above the base level of the footing, the effect of soil surrounding the footing above its base level is considered equivalent to a surcharge σ = γD.

FAILURE ZONES ACCORDING TO TERZAGHI:



Failure zones according to Terzaghi.

1. Zone I is elastic zone. When footing moves downward during failure, this zone moves downward along with footing. It behaves as though it is a part of the footing.
2. Zone II is radial shear zone bound by two planes inclined at Φ and $\left(45^0 - \frac{\emptyset}{2}\right)$ to the horizontal, and the base being a logarithmic spiral in section.
3. One set of planes in this zone radiate from a corner of the footing.
4. Zone III is linear shear zone or Rankine passive zone with failure planes inclined at $\left(45^0 - \frac{\emptyset}{2}\right)$ to the horizontal.

TERZAGHI BEARING CAPACITY EQUATION:

The Terzaghi's bearing capacity equation is given by:

$$q_u = c\,N_c + \gamma\,D\,N_q + 0.5\,\gamma B\,N_\gamma$$

- C = Cohesion
- q_u = Ultimate bearing capacity of the soil
- γ = Unit weight of soil
- N_C , N_q , N_γ = Bearing capacity factors
- D = Depth of foundation
- B = Breadth of foundation

LIMITATIONS TO TERZAGHI ANALYSIS:

1. Terzaghi's analysis assumes the plastic zones develop fully before failure occurs. This is true only in the case of dense cohesionless soils and stiff cohesive soils.
2. The value of Φ is assumed to remain constant. But Φ can change as soil gets compressed.
3. The failure zones are assumed not to extend above the base level of footing. Thus the shearing resistance of soil surrounding it above its base level is neglected.
4. The error due to this assumption increases as the depth of footing is increased.
5. The load is assumed to be vertical and acting concentrically with uniform pressure distribution at the base.

1.7 PENETRATION TESTS

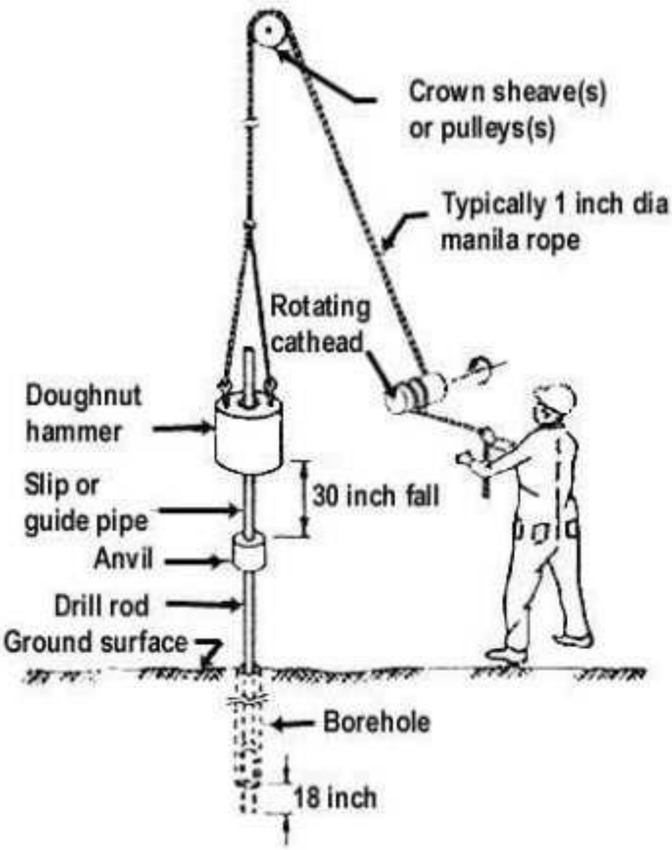
These tests involve the measurement of the resistance to penetration of a sampling spoon, a cone or other shaped tools under dynamic or static loadings. The resistance is empirically correlated with some of the engineering properties of soil such as density index, consistency, bearing capacity, etc., The values of these tests lie in the amount of experience behind h them. These tests are useful for general exploration of erratic soil profiles, for finding the depth to bed rock or hard stratum, and to have an approximate indication of the strength and other properties of soils, particularly the cohesionless soils, from which it is difficult to obtain undisturbed samples. The 2 commonly used tests are ; a) Standard penetration test & b)Cone penetration test.

1.7.1 STANDARD PENETRATION TESTS – IS 2131 : 1963

The standard penetration test is an in-situ test that is coming under the category of penetrometer tests. The standard penetration tests are carried out in borehole. The test will measure the resistance of the soil strata to the penetration undergone. A penetration emphirical correlation is derived between the soil properties and the penetration resistance. The test is extremely useful for determining the relative density and the angle of shearing resistance of cohesionless soils. It can also be used to determine the unconfined compressive strength of cohesive soils.

TOOLS FOR SPT:

1. Standard Split Spoon Sampler
2. Drop Hammer weighing 63.5kg
3. Guiding rod
4. Drilling Rig.
5. Driving head (anvil).



PROCEDURE

The test is conducted in a bore hole by means of a standard split spoon sampler. Once the drilling is done to the desired depth, the drilling tool is removed and the sampler is placed inside the bore hole. By means of a drop hammer of 63.5kg mass falling through a height of 750mm at the rate of 30 blows per minute, the sampler is driven into the soil. This is as per IS -2131:1963. The number of blows of hammer required to drive a depth of 150mm is counted. Further it is driven by 150 mm and the blows are counted. Similarly, the sampler is once again further driven by 150mm and the number of blows recorded. The number of blows recorded for the first 150mm not taken into consideration. The number of blows recorded for last two 150mm intervals are added to give the **standard penetration number (N)**. In other words,

N = No of blows required for 150mm penetration beyond seating drive of 150mm.

If the number of blows for 150mm drive exceeds 50, it is taken as refusal and the test is discontinued. The standard penetration number is corrected for dilatancy correction and overburden correction.

CORRECTIONS IN STANDARD PENETRATION TEST

Before the SPT values are used in empirical correlations and in design charts, the field 'N' value have to be corrected as per **IS 2131 – 1981.** The corrections are:

1. Dilatancy Correction
2. Overburden Pressure Correction

1. DILATANCY CORRECTION:

Silty fine sands and fine sands below the water table develop pore water pressure which is not easily dissipated. The pore pressure increases the resistance of the soil and hence the penetration number (N). Terzaghi and Peck (1967) recommend the following correction in the case of silty fine sands when the observed value is N exceeds 15. The corrected penetration number,

$$N_C = 15 + 0.5\,(N_R - 15)$$

N_R is the recorded value and N_C is the corrected value. If N_R less than or equal to 15, then $N_c = N_R$

2. OVERBURDEN PRESSURE CORRECTION:

From several investigations, it is proven that the penetration resistance or the value of **N** is dependent on the overburden pressure. If there are two granular soils with relative density same, higher 'N' value will be shown by the soil with higher confining pressure. With the increase in the depth of the soil, the confining pressure also increases. So the value of 'N' at shallow depth and larger depths are underestimated and overestimated respectively. Hence, to account this the value of 'N' obtained from the test are corrected to a standard effective overburden pressure. The corrected value of 'N' is

$$Nc = C_N\,N$$

Here $C_{N\ is}$ the correction factor for the overburden pressure.

PRECAUTIONS TAKEN FOR STANDARD PENETRATION TEST

1. Split spoon sampler must be in good condition.
2. The cutting shoe must be free from wear and tear
3. The height of fall must be 750mm. Any change from this will affect the 'N' value.
4. The drill rods used must be in standard condition. Bent drill rods are not used.
5. Before conducting the test, the bottom of the borehole must be cleaned.

ADVANTAGES OF STANDARD PENETRATION TEST

1. The test is simple and economical
2. The test provides representative samples for visual inspection, classification tests and for moisture content.
3. Actual soil behaviour is obtained through SPT values
4. The method helps to penetrate dense layers and fills
5. Test can be applied for variety of soil conditions

DIS-ADVANTAGES OF STANDARD PENETRATION TEST

1. The results will vary due to any mechanical or operator variability or drilling disturbances.
2. Test is costly and time consuming.
3. The samples retrieved for testing is disturbed.
4. The test results from SPT cannot be reproduced
5. The application of SPT in gravels, cobbles and cohesive soils are limited

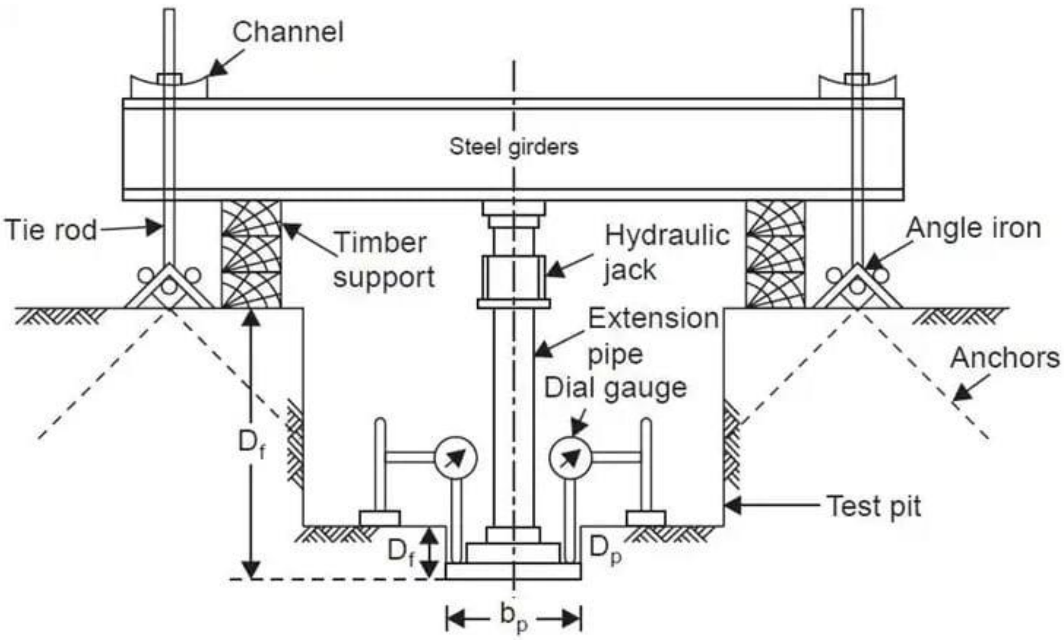
1.7.2 PLATE LOAD TEST – IS 2131 : 1963

Plate load test is done at site to determine the ultimate bearing capacity of soil and settlement of foundation under the loads for clayey and sandy soils. So, plate load test is helpful for the selection and design the foundation. To calculate safe bearing capacity suitable factor of safety is applied.

TOOLS FOR SPT:

- Mild Steel plate
- Hydraulic jack
- Reaction beam or reaction truss
- Dial gauges
- Excavating tools

PROCEDURE

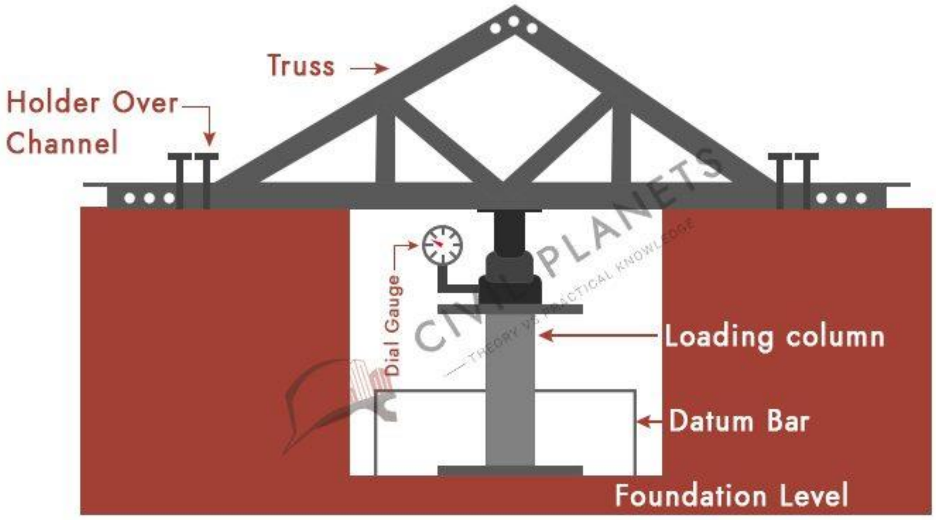


A Pit is excavated in the ground at which foundation is to be laid. The size of pit is generally 5 times the size of the plate. The depth excavated should be equal to proposed foundation depth. The plate used is made of mild steel. It may be square (0.3m x 0.3m) or circular (0.3m diameter) with 25mm thickness. After excavation of pit, at center of excavated pit steel plate sized hole with “Dp” thickness is excavated and arrange the plate in it. The thickness of plate Dp is calculated by below formula

$$\frac{\textbf{Thickness of Central Hole }(Dp)}{\textbf{Size of the Plate }(Bp)} = \frac{\textbf{Depth of Foundation }(Df)}{\textbf{Width of the Pit }(Bf)}$$

After arranging the plate in central hole hydraulic jack is arranged on top of plate to apply load. Reaction beam or reaction trusses is provided for the hydraulic jack to take up the reaction. Otherwise a loaded platform is created (using sand bags etc.) on the top of hydraulic jack and provided the reaction. After that seating load of 7kN/m2 is applied to set the plate and released after some time. Now load is applied with an increment of 20% of safe load. Dial gauges are arranged at bottom to record the settlement values. At 1min, 5min, 10min, 20min, 40min, and 60min and after that for every one-hour interval the settlement is observed and noted. The observations are made until the total settlement of 25mm has occurred.

In case of reaction truss loading, a truss is arranged on jack and both sides of truss are anchored to the ground with strong support. Two ends of truss are loaded uniformly, then truss transforms the load into the plate and settlement occurs. Load is applied with an increment of 2kN at every interval. Settlement is observed at different intervals as said in above method. For clayey soils, the observations are made until the rate of settlement is to be 0.2mm per hour.



CALCULATION OF BEARING CAPACITY FROM PLATE LOAD TEST:

From the results obtained in plate load test, a logarithmic graph is drawn between loads applied to the corresponding settlement. Load is taken on x-axis and settlement is on y-axis. From the graph, we can find out the value of ultimate load for the plate, which is equal to the corresponding settlement of 1/5th of plate width. The curve breaks at one point, the load corresponding to that break point is considered as ultimate load for plate. From this ultimate load for plate we can determine the value of ultimate bearing capacity and safe bearing capacity of soil for foundation.

Bearing Capacity calculation for Clayey soil,

Ultimate bearing capacity = Ultimate load for plate

$$q_u(f) = q_u(p)$$

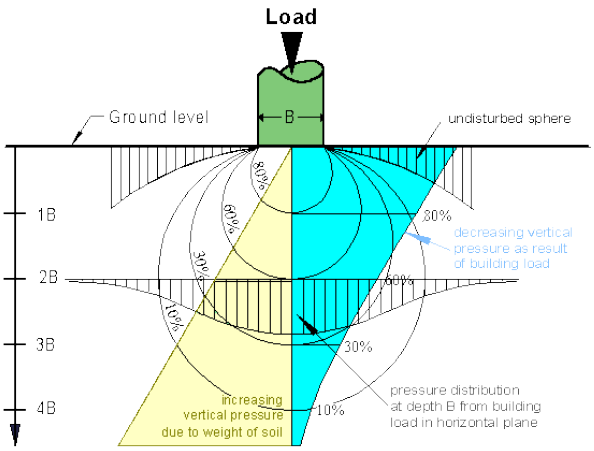
Bearing Capacity calculation for sandy soil,

$$\text{Ultimate bearing capacity} = \text{Ultimate load for plate x } \frac{\textit{Width of the pit}}{\textit{Size of plate}}$$

$$q_u(f) = q_u(p)\ x\ \frac{B_f}{B_p}$$

1.8 PRESSURE DISTRIBUTION IN DIFFERENT TYPES OF SOIL

The stability of structure is majorly depends upon soil – foundation interaction. Even though they are of different physical nature, they both must be act together to get required stability. So, It is important to know about the contact pressure developed between soil and foundation and its distribution in different conditions which is briefly explained below.



FOUNDATION CONTACT PRESSURE

Generally loads from the structure are transferred to the soil through footing. A reaction to this load, soil exerts an upward pressure on the bottom surface of the footing which is termed as contact pressure.

ASSUMPTIONS OF CONTACT PRESSURE DISTRIBUTION

- In the design of foundations, Contact pressure is assumed to be uniform which is not a problem for flexible foundations since they have uniform contact pressure irrespective of stiffness of soil.
- But when it comes to rigid foundation, this assumption may lead to unsafe design since contact pressure is not uniform in this case. This happens when the soil acts as elastic material.
- However, the soil under footing acts as elasto-plastic material just before failure occurs. Hence, this assumption can be justified at the ultimate stage.

1.8.1 FACTORS EFFECTING CONTACT PRESSURE DISTRIBUTION

Following are the factors effecting contact pressure distribution

- Stiffness of Footing
- Compressibility of soil
- Type of loading

STIFFNESS OF FOOTING

Contact pressure is uniform in case of flexible footings such as earth embankments. Contact pressure varies in case of rigid foundations such as R.C.C pad foundations etc. If the footing is partly flexible and partly rigid like raft foundation, contact pressure slightly varies.

COMPRESSIBILITY OF SOIL

Compressibility or stiffness of soil also plays a role in contact pressure distribution. If the soil is coarse grained, contact pressure is more at the center of foundation than edges where as in case of clayey soils contact pressure is uniform.

TYPE OF LOADING

Concentrated Loading :

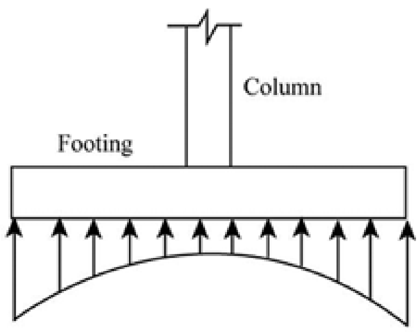
- If concentrated loading is applied at the center of foundation resting on cohesive soil, contact pressure is not uniform irrespective of stiffness of foundation.
- For flexible foundation, contact pressure is maximum exactly under the load application.
- For rigid foundations, contact pressure is maximum at edges. So, application of point load on rigid foundations can be comparable to the application of uniform loading on rigid foundation resting on cohesive soil.

Uniform Loading :

- Contact pressure distribution under uniform loading and deformed patterns of flexible and rigid foundations

1.8.2 PRESSURE DISTRIBUTION IN COHESIONLESS SOIL

- For footings on cohesive soil, edge stresses may be very large.
- However, pressure distribution may be considered to be linear for the purpose of design of reinforced cement concrete footing
- Once the pressure distribution is known, the bending moment and shear force in the footing can be calculated and the thickness of the structural member of footing alone with reinforcement etc, can be calculated using principles of RCC



Soil pressure distribution in cohesive soil.

1.8.3 PRESSURE DISTRIBUTION IN COHESIVE SOIL

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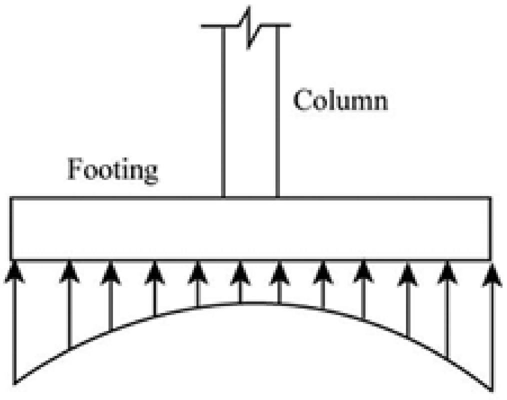
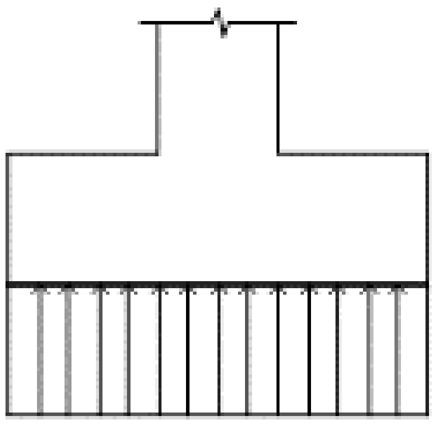


Soil pressure distribution in cohesive soil.



Uniform Pressure (Assumed condition)

UNIT - 2

BASIC MASONRY DESIGN

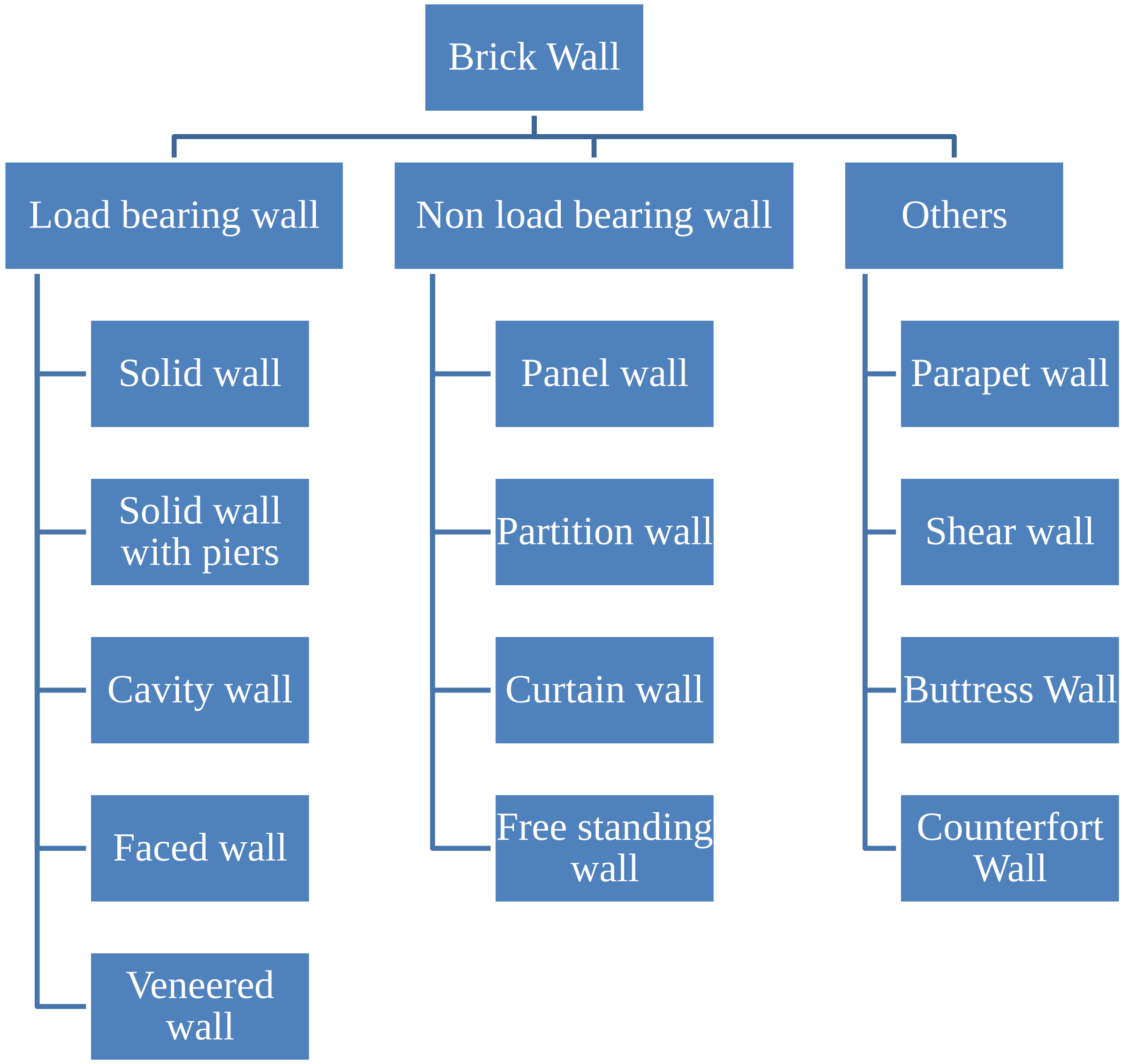
2.1 INTRODUCTION

Until 1950's there were no engineering methods of designing masonry for buildings and thickness of walls was being based on 'Rule-of-Thumb' Tables given in Building Codes and Regulations. As a result, walls used to be very thick and masonry structures were found to be very uneconomical beyond 3 or 4 storeys. Buildings exceeding 3 or 4 storeys had thus to be constructed with steel or RCC frames. Since 1950's intensive theoretical and experimental research has been conducted on various aspects of masonry in advanced countries. As a result, different factors which effect strength, stability and performance of masonry structures have been identified and methods of design based on engineering principles evolved. Most of the countries have therefore switched over to use of so called "calculated or engineering masonry" of structures. Simultaneously methods of manufacture of bricks and construction techniques have been considerably improved upon.

2.2 BRICK MASONRY WALLS

Brick masonry is a highly durable form of construction. It is built by placing bricks in mortar in a systematic manner to construct solid mass that withstand exerted loads. The bond in brick masonry, which adheres bricks together, is produced by filling joints between bricks with suitable mortar. A wall can be defined as an upright member, the width of which exceeds four times its thickness. If the ratio is less than four, the wall is considered as a column. Walls are of many types with various structural configurations and functions to serve.

2.3 CLASSIFICATION OF WALLS

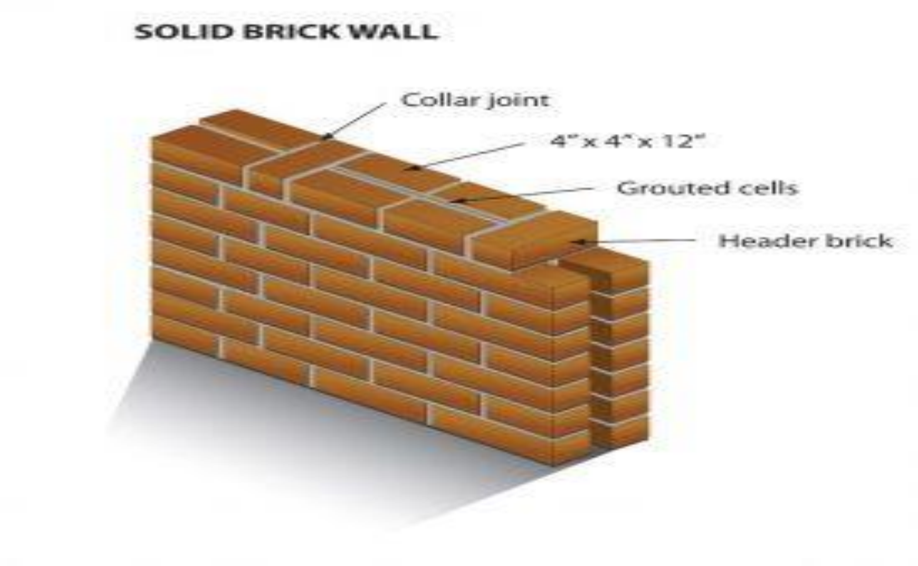


2.3.1 LOAD BEARING WALL

A load-bearing wall or bearing wall is a wall that is an active structural element of a building, which holds the weight of the elements above it, by conducting its weight to a foundation structure below it. Load-bearing walls are one of the earliest forms of construction.

2.3.1.1 SOLID WALL

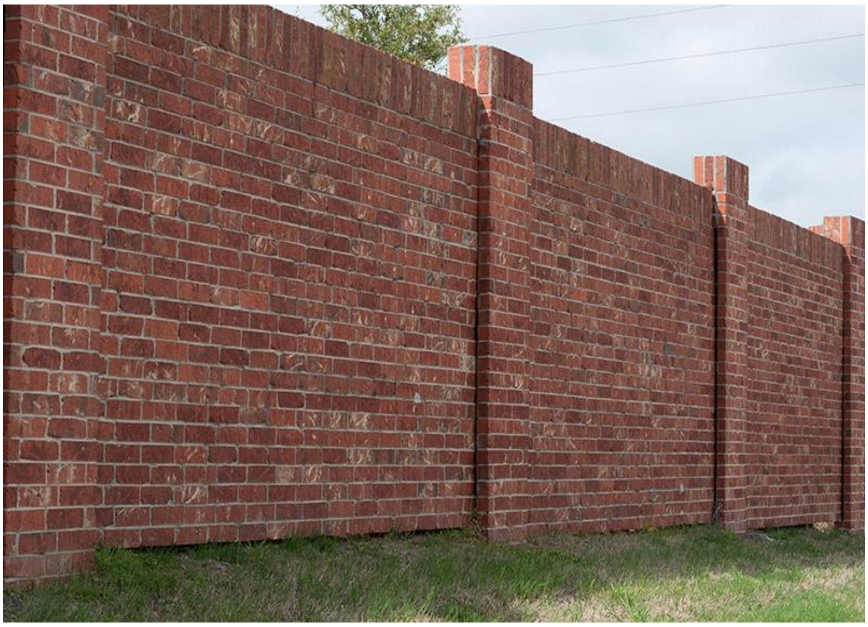
A solid wall is a wall built of solid or perforated brickwork and designed to carry an imposed load and its own weight.



2.3.1.2 SOLID WALL WITH PIERS

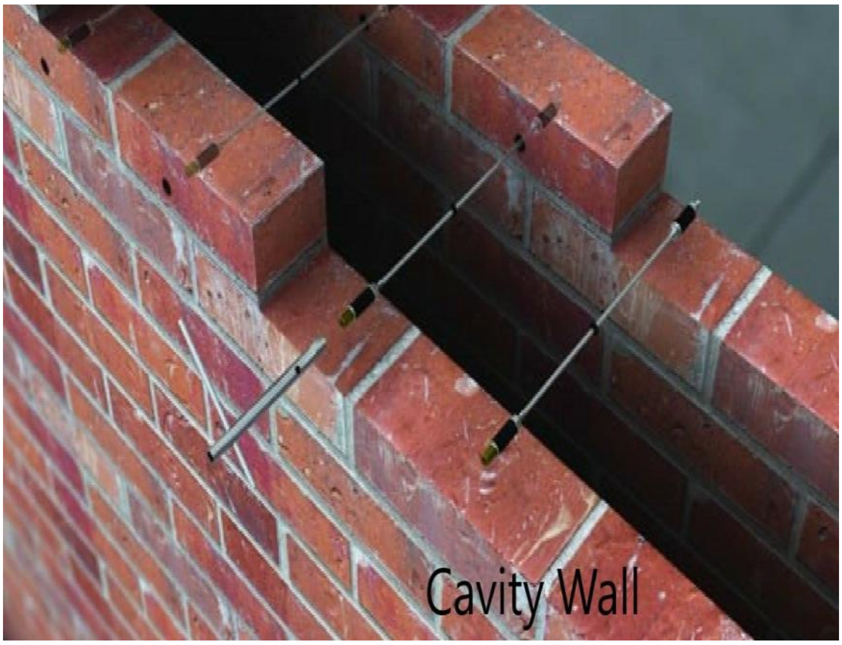
A solid wall with piers is a wall thickened at intervals by increasing the cross section. These thickened portions, known as Piers or Pilasters, are used for the following purposes:

- To carry concentrated loads from roof or floor beams
- To provide lateral support
- To stiffen the wall in order to lower the slenderness ratio within certain limits



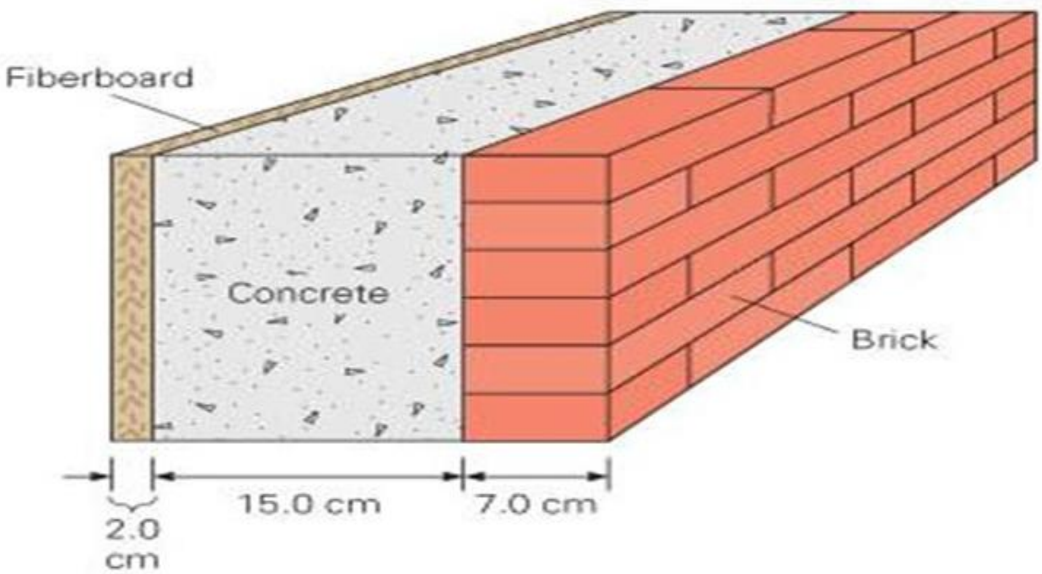
2.3.1.3 CAVITY WALL

A wall separated into 2 structural leaves by a continuous cavity with the two leaves being interconnected by metal ties. The cavity forms a barrier against penetration of dampness through to the internal wall face. Usually, the inner leaf is load bearing, while the outer leaf carries its own weight only. Bending moment is shared by stiffness of both the leaves



2.3.1.4 FACED WALL

A wall in which the facing and backing are made of two different materials and are bonded together to ensure a common action under the load



2.3.2 NON-LOAD BEARING WALL

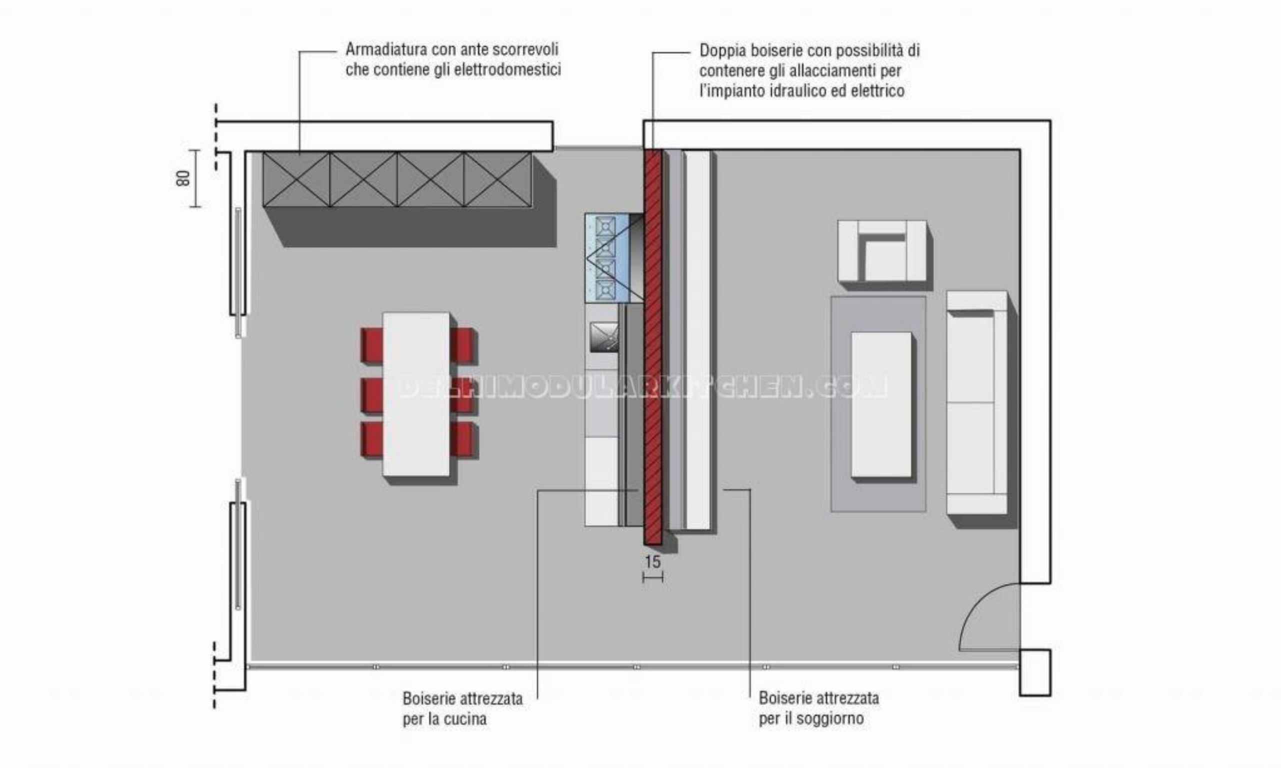
Non-load bearing walls are placed inside the house and do not support any structural weight of the building. They don't bear any additional weight of the house's structure other than their own. If the joists and rafters run parallel to the wall, they are often considered as non-load bearing walls.

2.3.2.1 PANEL WALL

An exterior non-load bearing wall in a structural framed construction and it forms into a unit in each storey. A wall panel is single piece of material, usually flat and cut into a rectangular shape, that serves as the visible and exposed covering for a wall. Holes may be cut or drilled into a wall panel to accommodate electrical outlets and other devices coming out of the wall.

2.3.2.2 PARTITION WALL

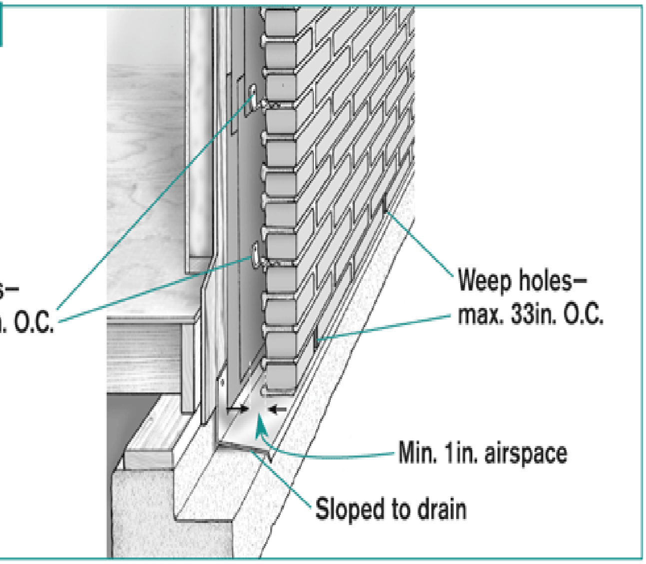
An interior non-load bearing wall, one storey or part storey in height used for partitioning a large room or hall



2.3.2.3 CURTAIN WALL

A self-supporting wall carrying no superimposed vertical load but subjected to lateral loads. It may be laterally supported by vertical or horizontal structural members wherever necessary

FIG. 12



2.3.3 OTHER WALL

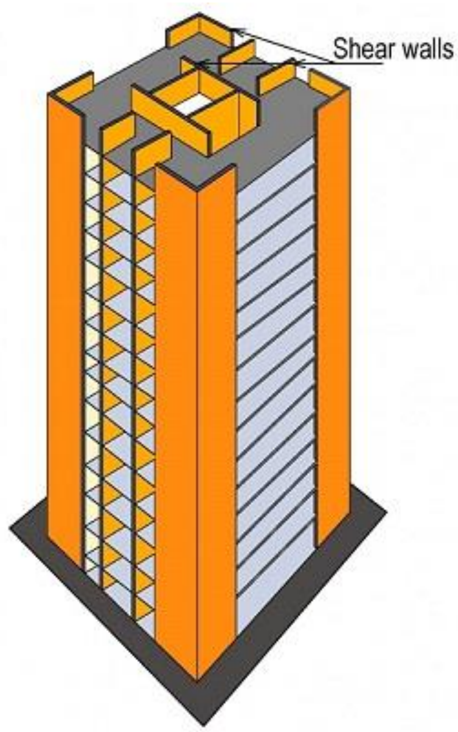
Walls other than Load bearing and Non-load bearing walls comes under the category of other walls.

2.3.3.1 PARAPET WALL

A parapet is a barrier that is an extension of the wall at the edge of a roof, terrace, balcony, walkway or other structure. Walls which do not carry any vertical imposed loads, but are expected to resist lateral wind load.

2.3.3.2 SHEAR WALL

A wall designed to carry primarily horizontal forces in its plane which may or may not have vertical imposed loads. A cross wall or a stiffening wall in a building functions as a shear wall. These are common in tall building construction

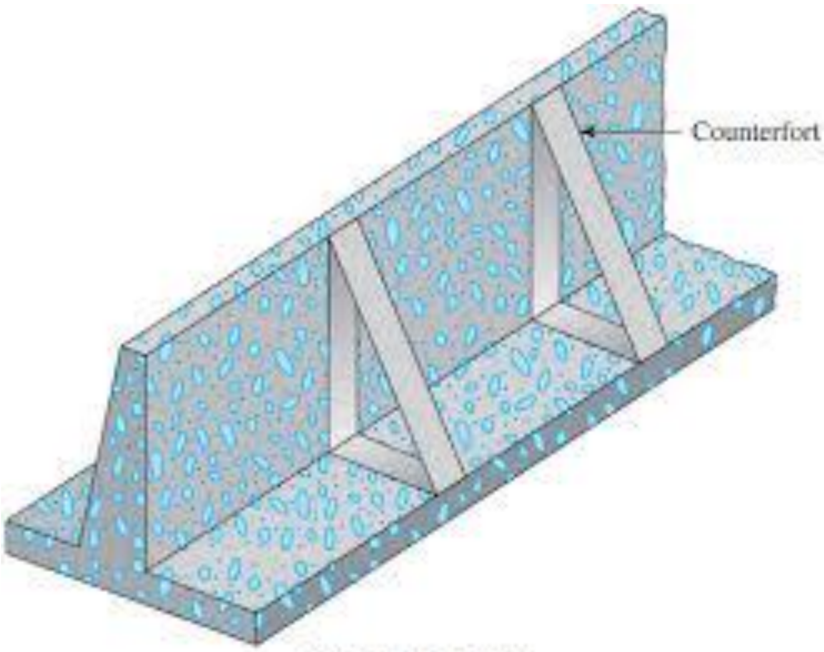


2.3.3.3 BUTTRESS WALL

A thickened portion of a wall which provides lateral support to the wall. A buttress is provided on the face opposite to the load face.

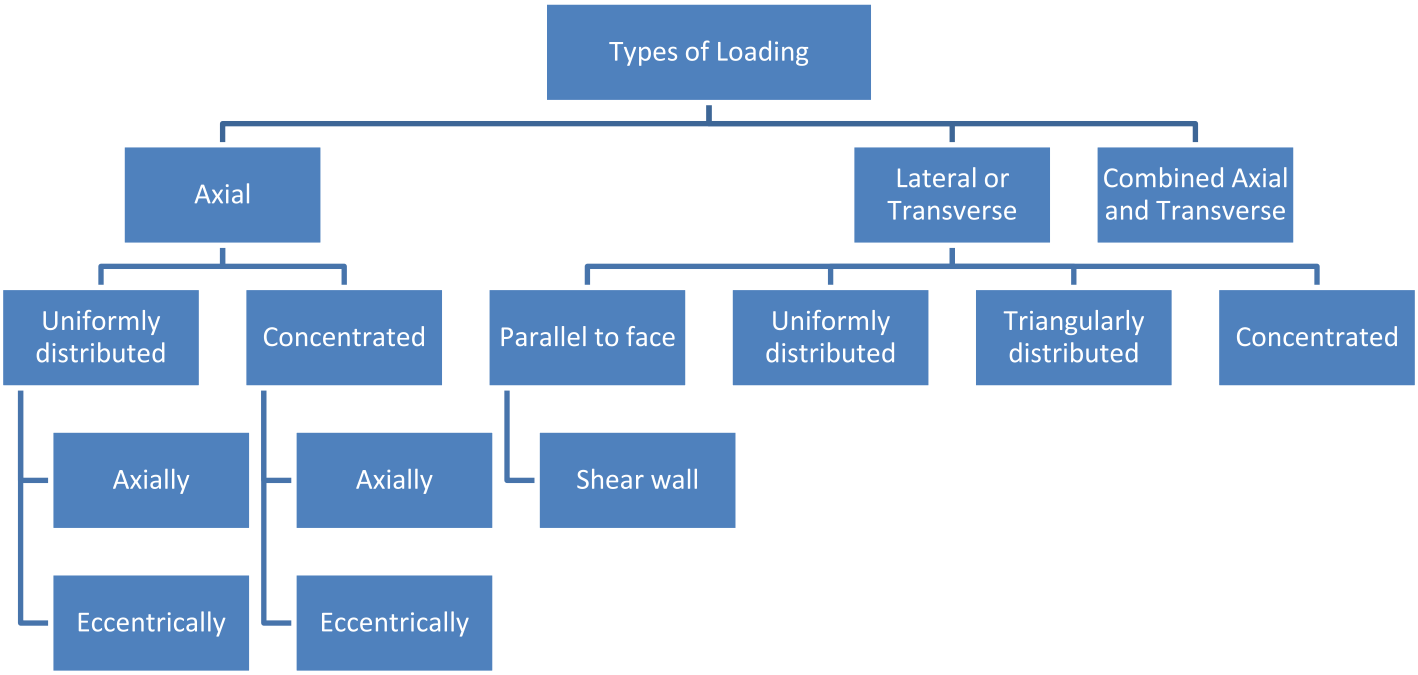
2.3.3.4 COUNTERFORT WALL

A thickened portion of wall which provides support to the wall and is located on the same face of the load.



(d) Counterfort wall

2.3.4 LOAD STRUCTURE INTERACTION AND CLASIFICATION



2.4 TECHNICAL SPECIFICATION OF A BRICK MASONRY WALL

- Boundary conditions
- Effective height
- End support conditions
- Effective length
- Effective thickness
- Slenderness ratio
- Stiffening coefficient
- Stress reduction factor
- Basic compressive stress

2.4.1 BOUNDARY CONDITIONS

Brick masonry walls have three main boundary conditions

- Full restraint – Both lateral and rotational restraints are offered by a support
- Partial restraint - Only lateral restraint is offered by a support
- No restraint – No lateral or rotational restraint is provided

2.4.2 EFFECTIVE HEIGHT OF WALL [CLAUSE 4.3 – PG:11 – IS 1905]

- The height of wall or column to be considered for calculating slenderness ratio.
- Effective height of wall is determined by the boundary conditions.

S.No	Types of Restraint		Effective Height
	Top	Bottom	
1	Full	Full	0.75 H
2(a)	Full	Partial	0.85 H
2(b)	Partial	Full	1.00 H
3	Partial	Partial	1.00 H
4	No	Full	1.50 H
5	No	Partial	2.00 H
*H = Actual height of wall between centers of adequate lateral support			

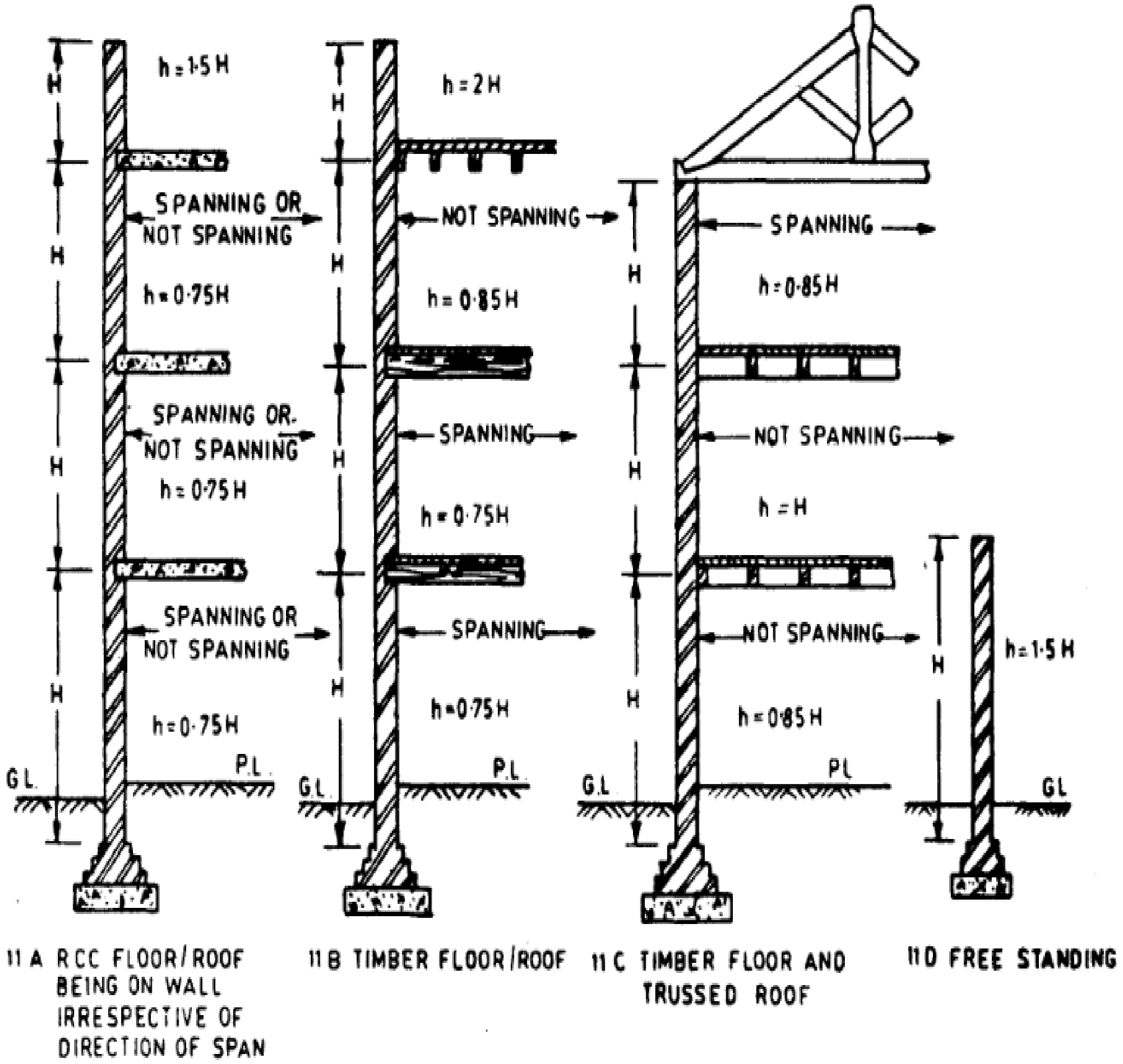


FIG. 11 EFFECTIVE HEIGHT OF WALL

2.4.3 END SUPPORT CONDITIONS

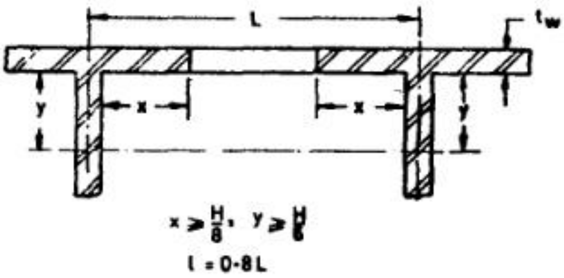
Brick masonry walls have three main end conditions

- Free end
- Continuity of wall
- Support from cross wall or pier or buttress
- Openings

2.4.4 EFFECTIVE LENGTH OF WALL [CLAUSE 4.4 – PG:12 – IS 1905]

- Effective length of wall to be considered for calculating slenderness ratio.
- Effective length of wall is determined by the end support conditions.

S.No	Condition of Support	Effective Length
1	Continuous wall, supported by cross wall / piers / buttress (no opening within a distance of H/8 from the face of cross wall)	0.8 L
2	Supported by a cross wall/Pier/Buttress at one end and continuous with cross wall/pier/buttress supporting at the other end	0.9 L
3	Supported at each end by a cross wall/pier/buttress	1.0 L
4	Free at one end and continuous with a cross wall/pier/buttress at other end	1.5 L
5	Free at one end and supported by a cross wall/pier/buttress at the other end	2.0 L
***L = Length of wall from or between center of cross wall/pier/buttress**		



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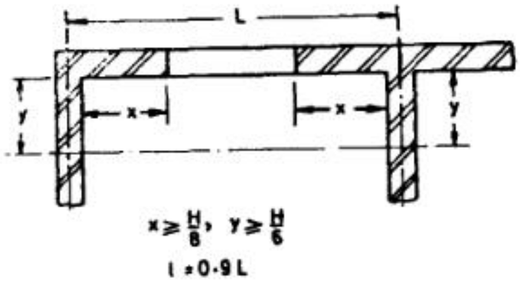

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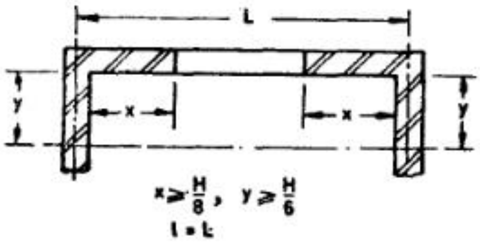

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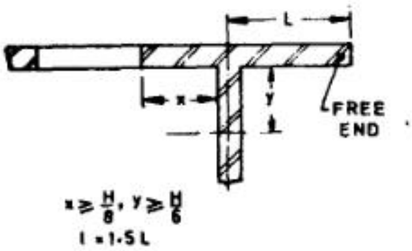

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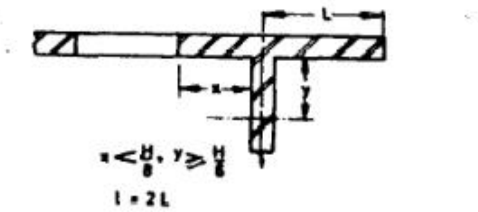

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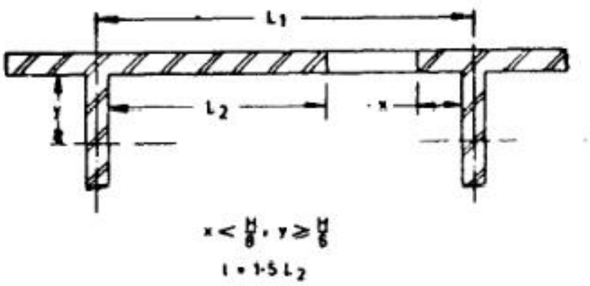

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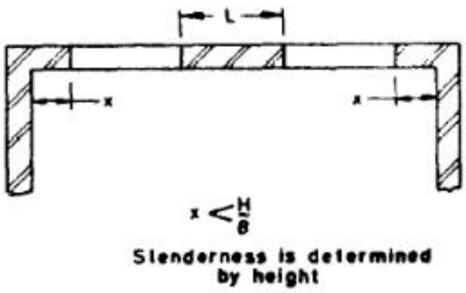


13G

2.4.5 EFFECTIVE THICKNESS OF WALL [CLAUSE 4.5 – PG:13 – IS 1905]

- Effective thickness of wall to be considered for calculating slenderness ratio.

S.No	Condition of Support	Effective Thickness
1	Solid and Faced Walls	t
2	Cavity walls – Both leaves of uniform thickness	$\frac{2}{3}(t_1 + t_2)$
3	Solid or Faced walls stiffened by cross walls / piers	Stiffening Coefficient * t

2.4.6 SLENDERNESS RATIO [CLAUSE 4.6 – PG:14 – IS 1905]

- Slenderness ratio is the ratio of effective height to Effective thickness or effective length to effective thickness

$$\text{Slenderness Ratio} = \frac{\text{Effective Height}}{\text{Effective Thickness}} \text{ or } \frac{\text{Effective Length}}{\text{Effective Thickness}}$$

Number of Storey's	Maximum Slenderness Ratio	
	Using Portland cement or Portland pozzolana cement in mortar	Using Lime Mortar
Not exceeding 2	27	20
Exceeding 2	27	13

2.4.7 STIFFENING COEFFICIENT [TABLE 6 – PG:14 – IS 1905]

TABLE 6 STIFFENING COEFFICIENT FOR WALLS STIFFENED BY PIERS, BUTTRESSES OR CROSS WALLS

(*Clauses* 4.5.2 *and* 4.5.3)

SL No.	S_P/w_P Ratio	Stiffening Coefficient		
		$t_P/t_w = 1$	$t_P/t_w = 2$	$t_P/t_w = 3$ or more
(1)	(2)	(3)	(4)	(5)
1	6	1·0	1·4	2·0
2	8	1·0	1·3	1·7
3	10	1·0	1·2	1·4
4	15	1·0	1·1	1·2
5	20 or more	1·0	1·0	1·0

where

S_P = centre-to-centre spacing of the pier or cross wall,

t_P = the thickness of pier as defined in **2.3.2** (*see* Fig. 1),

t_w = actual thickness of the wall proper (*see* Fig. 1), and

w_P = width of the pier in the direction of the wall or the actual thickness of the cross wall.

NOTE — Linear interpolation between the values given in this table is permissible but not extrapolation outside the limits given.

2.4.8 STRESS REDUCTION FACTOR [TABLE 9 – PG:16 – IS 1905]

TABLE 9 STRESS REDUCTION FACTOR FOR SLENDERNESS RATIO AND ECCENTRICITY

(*Clause* 5.4.1.1)

Slenderness Ratio	Eccentricity of Loading Divided by the Thickness of the Member					
	0	1/24	1/12	1/6	1/4	1/3
(1)	(2)	(3)	(4)	(5)	(6)	(7)
6	1·00	1·00	1·00	1·00	1·00	1·00
8	0·95	0·15	0·94	0·93	0·92	0·91
10	0·89	0·88	0·87	0·85	0·83	0·81
12	0·84	0·83	0·81	0·78	0·75	0·72
14	0·78	0·76	0·74	0·70	0·66	0·66
16	0·73	0·71	0·68	0·63	0·58	0·53
18	0·67	0·64	0·61	0·55	0·49	0·43
20	0·62	0·59	0·55	0·48	0·41	0·34
22	0·56	0·52	0·48	0·40	0·32	0·24
24	0·51	0·47	0·42	0·33	0·24	—
26	0·45	0·40	0·35	0·25	—	—
27	0·43	0·38	0·33	0·22	—	—

2.4.9 BASIC COMPRESSIVE STRESS [TABLE 8 – PG:16 – IS 1905]

TABLE 8 BASIC COMPRESSIVE STRESSES FOR MASONRY (AFTER 28 DAYS)

(*Clause* 5.4.1)

SL No.	MORTAR TYPE (REF TABLE 1)	BASIC COMPRESSIVE STRESSES IN N/mm² CORRESPONDING TO MASONRY UNITS OF WHICH HEIGHT TO WIDTH RATIO DOES NOT EXCEED 0·75 AND CRUSHING STRENGTH IN N/mm² IS NOT LESS THAN											
		3·5	5·0	7·5	10	12·5	15	17·5	20	25	30	35	40
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	H1	8·35	0·50	0·75	1·00	1·16	1·31	1·45	1·59	1·91	2·21	2·5	3·05
2	H2	8·35	0·50	0·74	0·96	1·09	1·19	1·30	1·41	1·62	1·85	2·1	2·5
3	M1	8·35	0·50	0·74	0·96	1·06	1·13	1·20	1·27	1·47	1·69	1·9	2·2
4	M2	0·35	0·44	0·59	0·81	0·94	1·03	1·10	1·17	1·34	1·51	1·65	1·9
5	M3	0·25	0.41	0·56	0·75	0·87	0·95	1·02	1·10	1·25	1·41	1·55	1.78
6	L1	0·25	0·36	0·53	0·67	0·76	0·83	0·90	0·97	1·11	1·26	1·4	1·06
7	L2	0·25	0·31	0·42	0·53	0·58	0·61	0·65	0·69	0·73	0·78	0·85	0·95

NOTE 1 — The table is valid for slenderness ratio up to 6 and loading with zero eccentricity.

NOTE 2 — The values given for basic compressive stress are applicable only when the masonry is properly cured.

NOTE 3 — Linear interpolation is permissible for units having crushing strengths between those given in the table.

NOTE 4 — The permissible stress for random rubble masonry may be taken as 75 percent of the corresponding stress for coarsed walling of similar materials.

NOTE 5 — The strength of ashlar masonry (natural stone masonry of massive type with thin joints) is closely related to intrinsic strength of the stone and allowable working stress in excess of those given in the table may be allowed for such masonry at the discretion of the designer.

2.5 DESIGN PROCEDURE OF UNREINFORCED BRICK MASONRY WALL

Step 1: Select Thickness for self-weight purposes

Step 2: Select the appropriate loads and their combinations.

The combinations are :

Dead load + Live load

Dead load + Live load + Seismic load

Dead load + Live load + Wind load

Dead load + Seismic load

Step 3: Compute Effective height

Step 4: Compute Effective length

Step 5: Compute Stiffening Coefficient and Effective thickness

Step 6: Compute Slenderness ratio

Step 7: Based of Slenderness ratio and Eccentricity calculate stress reduction factor

Step 8: Calculate Basic compressive stress

Step 9: Calculate Permissible compressive stress

Step 10: Check for safety

Solid Wall:

1. **Design an interior cross wall of a two storeyed building to carry 100 mm thick RCC slab with 3m ceiling height. The wall is unstiffened and it supports a 2.65m wide slab.**

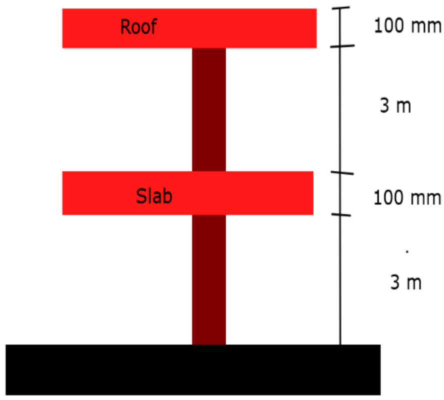
   Live load on roof = 1.5 kN/m^2

   Live load on floor = 2 kN/m^2

   Weight of 80 mm thick lime terrace = 1.96 kN/m^2

   Weight of floor finish = 0.8 kN/m^2

Given:



Thickness of Slab / roof = 100 mm

Height of each storey = 3000 mm

Width of slab = 2650 mm

Live load on roof = 1.5 kN/m^2

Live load on floor = 2 kN/m^2

Weight of 80 mm thick lime terrace = 1.96 kN/m^2

Weight of floor finish = 0.8 kN/m^2

Solution:

Step 1 : Assumed Data :

Thickness of masonry wall = 100 mm

Step 2 : Load Calculation :

Roof Load : Self weight of Roof = (1 x 1 x 0.1 x 25) = 2.5 kN/m^2

Weight of 80 mm thick lime terrace = 1.96 kN/m^2

Live load on roof = 1.5 kN/m^2

Total Load from roof = 5.96 kN/m^2

Floor Slab : Self weight of Slab = (1 x 1 x 0.1 x 25) = 2.5 kN/m^2

Weight of floor finish = 0.8 kN/m^2

Live load on floor = 2 kN/m^2

Total Load from Floor Slab = 5.3 kN/m^2

Wall : Self weight of Wall = 2 *(1 x 3 x 0.1 x 20) = 12 kN/m

Total Load of the structure = Weight of Slab and Roof + Weight of wall

= (5.96+5.3)*2.65 + 12

Total Load of the structure = 41.839 kN/m

Step 3 : Effective Height : (Table 4 – Pg 11)

Fully Restraint wall, Effective Height = 0.75 H = 0.75*3000 = 2250 mm

Step 4 : Effective Thickness : (Pg 13)

For solid walls, Effective thickness = Actual thickness = 100 mm

Step 5: Slenderness ratio : (Pg 14)

$$\textbf{Slenderness Ratio} = \frac{\textbf{Effective Height}}{\textbf{Effective Thickness}}$$

$$\text{Slenderness Ratio} = \frac{2250}{100} = 22.5 < 27.$$

Section is safe in slenderness.

Step 6 : Stress Reduction factor : (Table 9 - Pg 16)

For Slenderness ratio = 22.5 , Eccentricity = 0

SR	Ks
22	0.56
24	0.51

For SR = 22.5,

$$Ks = 0.56 + \frac{(0.51-0.56)}{(24-22)}(22.5-22)$$

$$k_s = 0.5475$$

Step 7 : Basic Compressive Stress : (Table 8 - Pg 16)

Assume Compressive strength of brick = 10 N/mm^2

Type of Mortar Used = M1

From Table 8, the Basic compressive stress of masonry = 0.96 N/mm^2

Step 8 : Permissible Compressive Stress :

Permissible Compressive stress = Ks * Basic Compressive stress

= 0.5475 * 0.96

Permissible Compressive stress = 0.5256 N/mm^2

Step 9 : Actual Compressive Stress :

Actual Compressive stress = Load / Area of wall

= (41.839 * 1000) / (1000*100)

Actual Compressive stress = 0.4183 N/mm^2 < 0.5256 N/mm^2

Selected Wall thickness of 100 mm is satisfactory.

Solid Wall supported at ends by Cross wall:

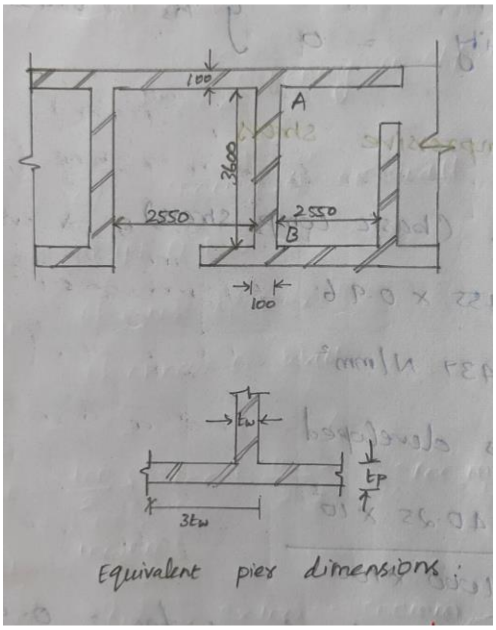
2. **Design an interior wall of a two storeyed building with RCC slabs of effective span 2.65 m. the wall is 3.6 m long and is stiffened at the ends by 100 mm thick intersecting walls. The ceiling height of each floor is 3m.**

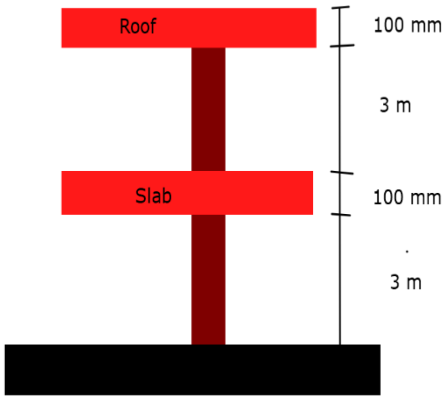
   Live load on roof = 1.5 kN/m^2

   Live load on floor = 2 kN/m^2

   Weight of 80 mm thick lime terrace = 1.96 kN/m^2

   Weight of floor finish = 0.8 kN/m^2





Given:

Thickness of Slab / roof = 100 mm (Assume)

Height of each storey = 3000 mm

Width of slab = 2650 mm

Live load on roof = 1.5 kN/m^2

Live load on floor = 2 kN/m^2

Weight of 80 mm thick lime terrace = 1.96 kN/m^2

Weight of floor finish = 0.8 kN/m^2

Wall End condition = Stiffened by cross walls

Length of the wall = 3600 mm

Solution:

Step 1 : Assumed Data :

Thickness of masonry wall = 100 mm

Step 2 : Load Calculation :

Roof Load : Self weight of Roof = (3.6 x 1 x 0.1 x 25) = 2.5 kN/m^2

Weight of 80 mm thick lime terrace = 1.96 kN/m^2

Live load on roof = 1.5 kN/m^2

Total Load from roof = 5.96 kN/m^2

Floor Slab : Self weight of Slab = (1 x 1 x 0.1 x 25) = 2.5 kN/m^2

Weight of floor finish = 0.8 kN/m^2

Live load on floor = 2 kN/m^2

Total Load from Floor Slab = 5.3 kN/m^2

Wall : Self weight of Wall = 2 *(1 x 3 x 0.1 x 20) = 12 kN/m

Total Load of the structure = Weight of Slab and Roof + Weight of wall

= (5.96+5.3)*2.65 + 12

Total Load of the structure = 41.839 kN/m

Step 3 : Effective Height : (Table 4 – Pg 11)

Fully Restraint wall, Effective Height = 0.75 H = 0.75*3000 = 2250 mm

Step 4 : Effective Length : (Table 5 – Pg 12)

Both ends are supported by cross wall, Effective Length = 1 L = 1*3600 = 3600 mm

Step 5: Effective Thickness : (Pg 13)

For solid walls stiffened at both ends by cross walls,

Effective thickness = Actual thickness * Stiffening Coefficient

S_p = Centre to centre spacing of cross wall = 50+3600+50 = 3700 mm

W_p = Width of the pier = 100 mm

t_p = Thickness of pier = 3 tw = 3*100 = 300mm

t_w = Thickness of wall = 100 mm

Sp/Wp = 3700/100 = 37

tp/tw = 300/100 = 3

From table 6 , Stiffening coefficient = 1

Effective thickness = 100 * 1 = 100 mm

Step 6: Slenderness ratio : (Pg 14)

$$\text{Slenderness Ratio} = \frac{\text{Effective Height}}{\text{Effective Thickness}} \text{ or } \frac{\text{Effective Length}}{\text{Effective Thickness}}$$

$$\text{Slenderness Ratio (Height)} = \frac{2250}{100} = 22.5 < 27.$$

$$\text{Slenderness Ratio (Length)} = \frac{3600}{100} = 36 > 27$$

Final Slenderness ratio = 22.5

Step 7 : Stress Reduction factor : (Table 9 - Pg 16)

For Slenderness ratio = 22.5 , Eccentricity = 0

SR	*Ks*
22	0.56
24	0.51

For SR = 22.5, $$Ks = 0.56 + \frac{(0.51-0.56)}{(24-22)}(22.5-22)$$

$$k_s = 0.5475$$

Step 8 : Basic Compressive Stress : (Table 8 - Pg 16)

Assume Compressive strength of brick = 10 N/mm^2

Type of Mortar Used = M1

From Table 8, the basic compressive stress of masonry = 0.96 N/mm^2

Step 9 : Permissible Compressive Stress :

Permissible Compressive stress = Ks * Basic Compressive stress

= 0.5475 * 0.96

Permissible Compressive stress = 0.5256 N/mm^2

Step 10 : Actual Compressive Stress :

Actual Compressive stress = Load / Area of wall

= (41.839 * 1000) / (1000*100)

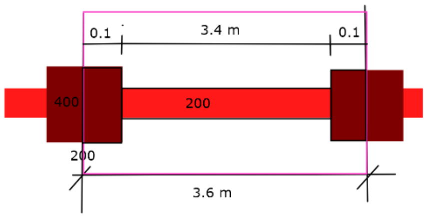
Actual Compressive stress = 0.4183 N/mm^2 < 0.5256 N/mm^2

Selected Wall thickness of 100 mm is satisfactory.

Solid Wall with piers:

3. **Design an interior wall of a single storeyed workshop of height 5.4m supporting a RCC roof. The bottom of wall rest over a foundation block. Assume roof load equal to 45 kN/m.**

Given:



Height of the workshop = 5.4m = 5400 mm

Roof load = 45 kN/m

Solution:

Step 1 : Assumed Data :

Thickness of masonry wall = 200 mm

Width of the pier = 200 mm

Thickness of pier = 400 mm

Spacing between the piers = 3600 mm c/c

Step 2 : Load Calculation :

Load from the roof	=	45 kN/m * 3.6	=	162 kN
Self weight of the wall	=	3.4 * 5.4 * 0.2 * 20	=	73.44 kN
Self weight of piers	=	(0.1*0.4*5.4*20)*2	=	8.64 kN
		Total Load on foundation =		244 kN

Step 3 : Effective Height : (Table 4 – Pg 11)

Fully Restraint wall, Effective Height = 0.75 H = 0.75*5400 = 4050 mm

Step 4 : Effective Length : (Table 5 – Pg 12)

Wall is continuous and supported by piers,

Effective Length = 0.8 L = 0.8*3600 = 2880 mm

Step 5: Effective Thickness : (Pg 13)

For solid walls stiffened at both ends by piers,

Effective thickness = Actual thickness * Stiffening Coefficient

S_p = Centre to centre spacing of piers = 3.6m = 3600 mm

W_p = Width of the pier = 200 mm

t_p = Thickness of pier = 400 mm

t_w = Thickness of wall = 200 mm

Sp/Wp = 3600/200 = 18

tp/tw = 400/200 = 2

From table 6 ,

Sp/wp	tp/tw
15	1.1
20	1.0

For Sp/Wp = 18, $Stiffening\ Coefficient = 1.1 + \frac{(1-1.1)}{(20-15)}(18-15)$

$$Stiffening\ Coefficient = 1.04$$

Effective thickness = 200 * 1.04 = 208 mm

Step 6: Slenderness ratio : (Pg 14)

$$\text{Slenderness Ratio} = \frac{\text{Effective Height}}{\text{Effective Thickness}}\ or\ \frac{\text{Effective Length}}{\text{Effective Thickness}}$$

$\text{Slenderness Ratio (Height)} = \frac{4050}{208}$ = 19.47 = 20 < 27.

$\text{Slenderness Ratio (Length)} = \frac{2880}{208}$ = 13.85 = 14 < 27

Final Slenderness ratio = 14

Step 7 : Stress Reduction factor : (Table 9 - Pg 16)

For Slenderness ratio = 14 , Eccentricity = 0 $k_s = 0.78$

Step 8 : Basic Compressive Stress : (Table 8 - Pg 16)

Assume Compressive strength of brick = 10 N/mm^2

Type of Mortar Used = M1

From Table 8, the basic compressive stress of masonry = 0.96 N/mm^2

Step 9 : Permissible Compressive Stress :

Permissible Compressive stress = Ks * Basic Compressive stress

= 0.78 * 0.96

Permissible Compressive stress = 0.748 N/mm^2

Step 10 : Actual Compressive Stress :

Actual Compressive stress σ_c = $\frac{Load}{Cross\ sectional\ Area\ of\ the\ wall}$

Cross sectional Area of the wall = (100*400)*2 +(3400*200) = 760000 mm^2

Actual Compressive stress σ_c = $\frac{244*1000}{760000}$

Actual Compressive stress = 0.321 N/mm^2 < 0.748 N/mm^2

Actual Compressive stress < Permissible compressive stress

Selected Wall thickness of 200 mm and pier 200 x 400 mm is satisfactory.

Cavity Walls:

4.Design an interior cavity wall for a three storeyed building. The ceiling height of each storey being 3m. the wall is unstiffened and is 3.6m long. Assume the following data. Load from roof = 12 kN/m and load from floor = 10 kN/m for each floor.

Given:

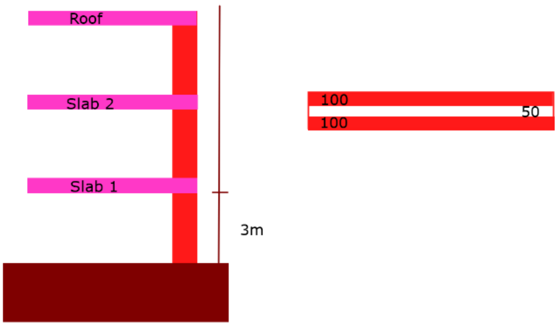
Height of the wall = 3m = 3000 mm

Length of the wall = 3.6 m = 3600 mm

Load from roof = 12 kN/m

Load from floor = 10 kN/m

Wall is unstiffened



Solution:

Step 1 : Assumed Data :

Overall Thickness of cavity masonry wall	= 250 mm
Thickness of each leaf	= 100 mm
Thickness of cavity	= 50 mm

Step 2 : Load Calculation :

Load from the roof		=	12 kN/m
Load from Slab 2		=	10 kN/m
Load from slab 1		=	10 kN/m
Self weight of the wall	=3 * (2 * (1 * 3 * 0.1 * 20))	=	36 kN/m
	Total Load of the structure	=	68 kN/m

Step 3 : Effective Height : (Table 4 – Pg 11)

Fully Restraint wall, Effective Height = 0.75 H = 0.75*3000 = 2250 mm

Step 4 : Effective Length : (Table 5 – Pg 12)

Effective Length = 1.0 L = 1*3600 = 3600 mm

Step 5: Effective Thickness : (Pg 13&14)

For cavity Walls , Effective thickness = $\frac{2}{3}(t_1 + t_2)$

t_1 = 100 mm t_2 = 100 mm

Effective thickness = $\frac{2}{3}(100 + 100)$

Effective thickness = 133.3 mm

Step 6: Slenderness ratio : (Pg 14)

$$\text{Slenderness Ratio} = \frac{\text{Effective Height}}{\text{Effective Thickness}} \text{ or } \frac{\text{Effective Length}}{\text{Effective Thickness}}$$

Slenderness Ratio (Height) = $\frac{2250}{133.3}$ = 16.87 = 17

Slenderness Ratio (Length) = $\frac{3600}{133.3}$ = 27 = 27

Final Slenderness ratio = 17 < 27

Step 7 : Stress Reduction factor : (Table 9 - Pg 16)

For Slenderness ratio = 17 , Eccentricity = 0

For SR = 17, $Ks = 0.73 + \frac{(0.67-0.73)}{(18-16)}(17-16)$

$k_s = 0.7$

SR	*Ks*
16	0.73
18	0.67

Step 8 : Basic Compressive Stress : (Table 8 - Pg 16)

Assume Compressive strength of brick = 7.5 N/mm^2

Type of Mortar Used = M1

From Table 8, the basic compressive stress of masonry = 0.74 N/mm^2

Step 9 : Permissible Compressive Stress :

Permissible Compressive stress = Ks * Basic Compressive stress

= 0.7 * 0.74

Permissible Compressive stress = 0.518 N/mm^2

Step 10 : Actual Compressive Stress :

Actual Compressive stress σ_c = $\frac{Load}{Cross\ sectional\ Area\ of\ the\ wall} = \frac{Load}{b*t}$

Actual Compressive stress σ_c = $\frac{68*1000}{1000*200}$

Actual Compressive stress = 0.34 N/mm^2 < 0.518 N/mm^2

Actual Compressive stress < Permissible compressive stress

Selected Wall thickness of 250 mm with each leaf 100mm & cavity of 50mm is satisfactory.

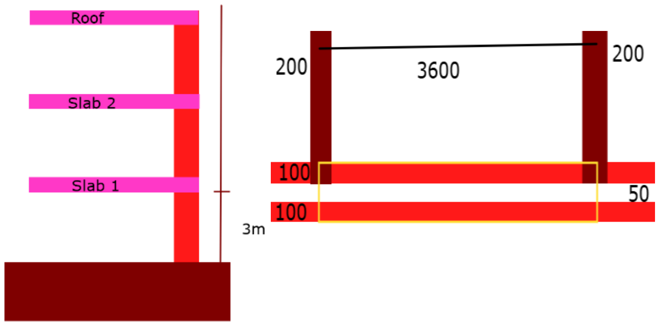
Homework:

5. Design an interior cavity wall of a three storeyed building, the ceiling height of each floor is 3.2m. the wall is unstiffened and is 3.5m long. Assume loading from roof as 10kN/m and from floors as 9kN/m.

Cavity Walls with Cross Walls:

6. Design an interior cavity wall with cross walls of a three storeyed building, the ceiling height of each storey being 3m. The wall is stiffened by intersecting walls 200mm thick at 3600 mm centre to centre. Assume roof loading = 16 kN/m and from each floor = 12.5 kN/m.

Given:



Height of the wall = 3m = 3000 mm

Length of the wall = 3.6 m = 3600 mm

Load from roof = 16 kN/m

Load from floor = 12.5 kN/m (each)

Wall is Stiffened

Stiffening – Thickness of intersecting wall = 200 mm and spacing is 3600 mm

Solution:

Step 1 : Assumed Data :

Overall Thickness of cavity masonry wall	=	250 mm
Thickness of each leaf	=	100 mm
Thickness of cavity	=	50 mm

Step 2 : Load Calculation :

Load from the roof		=	16 kN/m
Load from Slab 2		=	12.5 kN/m
Load from slab 1		=	12.5 kN/m
Self weight of the wall	=3 * (2 * (1 * 3 * 0.1 * 20))	=	36 kN/m
	Total Load of the structure	=	77 kN/m

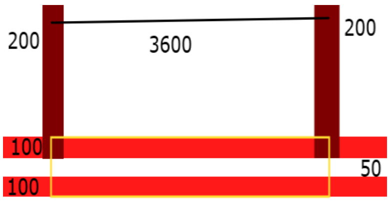
Step 3 : Effective Height : (Table 4 – Pg 11)

Fully Restraint wall, Effective Height = 0.75 H = 0.75*3000 = 2250 mm

Step 4 : Effective Length : (Table 5 – Pg 12)

Effective Length = 0.8 L = 0.8*3600 = 2880 mm

Step 5: Effective Thickness : (Pg 13&14)



For cavity Walls , Effective thickness = $\frac{2}{3}\ (t_1 + t_2)$

First Leaf – (Solid wall with cross wall)

Effective thickness = Actual thickness * Stiffening Coefficient

S_p = Centre to centre spacing of cross wall = 3600 mm

W_p = Width of the cross wall = 200 mm

t_p = Thickness of pier = 3 tw = 3*100 = 300mm

t_w = Thickness of wall = 100 mm

Sp/Wp = 3600/200 = 18

tp/tw = 300/100 = 3

Sp/wp	SC
15	1.2
20	1.0

For Sp/Wp = 18, $SC = 1.2 + \frac{(1.0-1.2)}{(20-15)}\ (18 - 15)$ $SC = 1.08$

Effective thickness of Leaf 1 (t1)= 100 * 1.08 = 108 mm

Second Leaf – (Solid wall)

Effective thickness of Leaf 2 (t2) = Actual thickness = 100 mm

For cavity Walls , Effective thickness = $\frac{2}{3}\ (t_1 + t_2)$

Effective thickness = $\frac{2}{3}\ (108 + 100)$

Effective thickness = 139 mm

Step 6: Slenderness ratio : (Pg 14)

$$\text{Slenderness Ratio} = \frac{\text{Effective Height}}{\text{Effective Thickness}} \text{ or } \frac{\text{Effective Length}}{\text{Effective Thickness}}$$

$$\text{Slenderness Ratio (Height)} = \frac{2250}{139} = 16.1 = 16$$

$$\text{Slenderness Ratio (Length)} = \frac{2880}{139} = 20.7 = 21$$

Final Slenderness ratio = 16 < 27

Step 7 : Stress Reduction factor : (Table 9 - Pg 16)

For Slenderness ratio = 16 , Eccentricity = 0 $k_s = 0.73$

Step 8 : Basic Compressive Stress : (Table 8 - Pg 16)

Assume Compressive strength of brick = 10 N/mm^2

Type of Mortar Used = M1

From Table 8, the Basic compressive stress of masonry = 0.96 N/mm^2

Step 9 : Permissible Compressive Stress :

Permissible Compressive stress = Ks * Basic Compressive stress

= 0.73 * 0.96

Permissible Compressive stress = 0.70 N/mm^2

Step 10 : Actual Compressive Stress :

$$\text{Actual Compressive stress } \sigma_c = \frac{Load}{Cross\ sectional\ Area\ of\ the\ wall} = \frac{Load}{b*t}$$

$$\text{Actual Compressive stress } \sigma_c = \frac{77*1000}{1000*200}$$

Actual Compressive stress = 0.385 N/mm^2 < 0.7 N/mm^2

Actual Compressive stress < Permissible compressive stress

Selected Wall thickness of 250 mm with each leaf 100mm & cavity of 50mm stiffened by 200 mm cross wall is satisfactory.

Solid Walls under Concentrated loads:

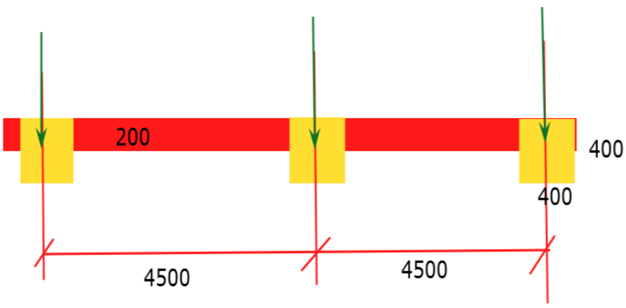
7. Design an exterior wall of a workshop building 3.6m high carrying steel trusses at the top at 4.5m spacing. The wall is securely tied at the roof and floor level. The loading shall be assumed as follows:

Concentrated reaction from roof trusses = 30kN acting at centre of wall

Roof loading = 7kN/m

Ignore wind loading

Given:



Height of the wall = 3.6m = 3600 mm

Length of the wall = 4.5 m = 4500 mm

Concentrated load from truss = 30 kN

Load from roof = 7 kN/m

Wall is Stiffened

Solution:

Step 1 : Assumed Data :

Thickness of wall = 200 mm

Stiffening – Thickness of piers = 400 mm

Width of the piers = 400 mm

Spacing of the piers = 4500 mm

Bearing Area = 200 x 200 mm

Step 2 : Load Calculation :

Because of load dispersion, the concentrated load area acts on a length of "l"

$$l = \frac{2H}{\tan 60^0} = \frac{2 * 3600}{\tan 60^0} = 4157\ mm$$

Length of the bay under consideration, l = 4157 mm

Load from the roof	= 7 * 4.157	=	29.1 kN
Load from Truss		=	30 kN
Self weight of the wall	= (4.157*3.6*0.2*20)	=	59.86 kN
Self-weight of piers	= (0.4*0.4*3.6*20)	=	11.52 kN
	Total Load of the structure	=	130.48 kN

Step 3 : Effective Height : (Table 4 – Pg 11)

Fully Restraint wall, Effective Height = 0.75 H = 0.75*3600 = 2700 mm

Step 4 : Effective Length : (Table 5 – Pg 12)

Effective Length = 0.8 L = 0.8*4157 = 3325.6 mm

Step 5: Effective Thickness : (Pg 13&14)

For walls stiffened with piers, Effective thickness = Actual thickness * Stiffening Coefficient

S_p = Centre to centre spacing of piers = 4500 mm

W_p = Width of the pier= 400 mm

t_p = Thickness of pier = 400mm

t_w = Thickness of wall = 200 mm

Sp/wp	SC
10	1.2
15	1.1

Sp/Wp = 4500/400 = 11.25 = 11

tp/tw = 400/200 = 2

For Sp/Wp = 11, $SC = 1.2 + \frac{(1.1-1.2)}{(15-10)}(11-10)$ $SC = 1.18$

Effective thickness = 200 * 1.18 = 236 mm

Step 6: Slenderness ratio : (Pg 14)

$$\text{Slenderness Ratio} = \frac{\text{Effective Height}}{\text{Effective Thickness}} \ or \ \frac{\text{Effective Length}}{\text{Effective Thickness}}$$

$\text{Slenderness Ratio (Height)} = \frac{2700}{236} = 11.44 = 12$

$\text{Slenderness Ratio (Length)} = \frac{3325.6}{236} = 14$

Final Slenderness ratio = 12 < 20 (Lime mortar)

Step 7 : Stress Reduction factor : (Table 9 - Pg 16)

For Slenderness ratio = 12 , Eccentricity = 0 $k_s = 0.84$

Step 8 : Basic Compressive Stress : (Table 8 - Pg 16)

Assume Compressive strength of brick = 10 N/mm^2

Type of Mortar Used = L1

From Table 8, the Basic compressive stress of masonry = 0.67 N/mm^2

Step 9 : Permissible Compressive Stress :

Permissible Compressive stress = Ks * Basic Compressive stress

= 0.84 * 0.67

Permissible Compressive stress = 0.5628 N/mm^2

Step 10 : Actual Compressive Stress :

Actual Compressive stress σ_c $= \frac{Load}{Cross\ sectional\ Area\ of\ the\ wall} =$

Actual Compressive stress σ_c $= \frac{130.48*1000}{(4157*200+400*400)}$

Actual Compressive stress = 0.132 N/mm^2 < 0.568 N/mm^2

Actual Compressive stress < Permissible compressive stress

Selected Wall thickness of 2000 mm is satisfactory.

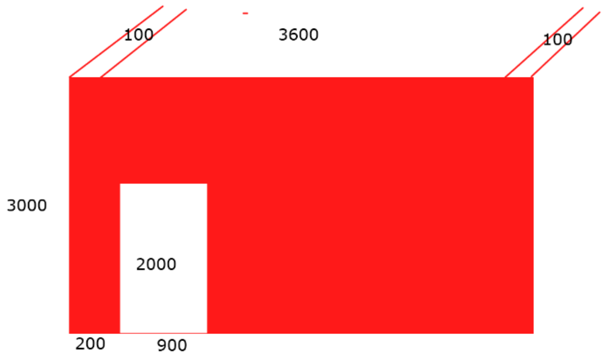
Solid Walls with openings:

8. Design an interior wall of a 2 storeyed building carrying concrete slab with a storey height 3m. The wall is stiffened by 100 mm thick intersecting walls at 3600 mm c/c. Also the wall has a door opening of size 900*2000 mm at a distance of 200 mm from one of the intersection walls. Assume loading as follows:

Roof loading = 15 kN/m

Floor loading = 12.5 kN/m

Given:



Height of the wall = 3.0m = 3000 mm

Length of the wall = 3.6 m = 3600 mm

Load from roof = 15 kN/m

Load from floor = 12.5 kN/m

Wall is Stiffened

Thickness of cross wall = 100mm

Spacing between the cross wall = 3600 mm

Door opening = 900 x 2000 mm

Solution:

Step 1 : Assumed Data :

Thickness of wall = 200 mm

Step 2 : Load Calculation :

Load from the roof	=	=	15 kN/m
Load from floor		=	12.5 kN/m
Self weight of the wall	= (1*3*0.2*20)*2	=	24 kN/m
	Total Load of the structure	=	51.5 kN/m

Step 3 : Effective Height : (Table 4 – Pg 11)

Fully Restraint wall, Effective Height = 0.75 H = 0.75*3000 = 2250 mm

Step 4 : Effective Length : (Table 5 – Pg 12)

Effective Length = 0.8 L = 0.8*3600 = 2880 mm

Step 5: Effective Thickness : (Pg 13&14)

For walls with openings,

$$\textit{Effective thickness of wall at the level of opening} = \frac{(L-a)t}{L}$$

$$\text{Effective thickness of wall at the level of opening} = \frac{(3600-900)200}{3600}$$

$$\text{Effective thickness of wall at the level of opening} = 150 \text{ mm}$$

$$\textit{Average thickness of wall} = \frac{\textit{Actual thickness} + \textit{Thickness at opening}}{2}$$

$$\text{Average thickness of wall} = \frac{200+150}{2} = 175\ mm$$

Step 6: Slenderness ratio : (Pg 14)

$$\textbf{Slenderness Ratio} = \frac{\text{Effective Height}}{\text{Effective Thickness}} \ \textit{or} \ \frac{\text{Effective Length}}{\text{Effective Thickness}}$$

$$\text{Slenderness Ratio (Height)} = \frac{2250}{175} = 12.86 = 13$$

$$\text{Slenderness Ratio (Length)} = \frac{2880}{175} = 16.45 = 17$$

Final Slenderness ratio = 13 < 27

Step 7 : Stress Reduction factor : (Table 9 - Pg 16)

For Slenderness ratio = 13 , Eccentricity = 0 $k_s = 0.81$

Step 8 : Basic Compressive Stress : (Table 8 - Pg 16)

Assume Compressive strength of brick = 10 N/mm^2

Type of Mortar Used = M1

From Table 8, the Basic compressive stress of masonry = 0.96 N/mm^2

Step 9 : Permissible Compressive Stress :

Permissible Compressive stress = Ks * Basic Compressive stress

= 0.81 * 0.96

Permissible Compressive stress = 0.7776 N/mm^2

Step 10 : Actual Compressive Stress :

Actual Compressive stress σ_c $= \frac{Load}{Cross\ sectional\ Area\ of\ the\ wall} = \frac{Load}{(L-a)t}$

Actual Compressive stress σ_c $= \frac{51.5\ x\ 1000}{(3600-900)200}$

Actual Compressive stress = 0.34 N/mm^2 < 0.7776 N/mm^2

Actual Compressive stress < Permissible compressive stress

Selected Wall thickness of 200 mm is satisfactory.

2.6 NOMOGRAMS

Nomograms are a pictorial representation of a complex mathematical formula. A nomogram, also called a nomograph, alignment chart, or abaque, is a graphical calculating device, a two-dimensional diagram designed to allow the approximate graphical computation of a mathematical function. A nomogram consists of a set of n scales, one for each variable in an equation. Knowing the values of n-1 variables, the value of the unknown variable can be found, or by fixing the values of some variables, the relationship between the unfixed ones can be studied. The result is obtained by laying a straightedge across the known values on the scales and reading the unknown value from where it crosses the scale for that variable. The virtual or drawn line created by the straightedge is called an index line or isopleth.

2.6.1 STRUCTURE OF NOMOGRAMS

The nomograms for thickness of brick masonry wall consist of nine-vertical lines. From left to right, the vertical lines represent,

- The basic stress,
- Storeys,
- Reference line 1,
- Span point,
- Reference line 2,
- Percentage of openings,
- Thickness of walls for spans of 3m,
- Thickness of walls for spans of 3.6m,
- Thickness of walls for spans of 4.2 m.

BASIC STRESS

- The basic stress of masonry, depending on the crushing strength of masonry unit (brick) and mortar used is indicated on the first vertical line.
- Table 1 of SP 10-1973 gives the basic stress for known values, of crushing strength of the masonry unit and the mortar used.
- Linear interpolation between the limits is permitted.

STOREYS

- The second line lists the number of storeys of the masonry building for which the thicknesses of brick wall are available.
- Masonry thicknesses are arrived at for buildings up to six storeys in height.
- For use of nomograms in the case of multi-storeyed buildings, the wall thickness at each floor is found by passing the line through the number of storeys above that section.
- For example, in a four-storeyed building the thickness of wall at the ground floor (Floor 1) is found by passing the line through ‘ 4 ’ on the storey line.
- Similarly, for Floor 2, the line shall be passed through ‘ 3 ’ on the storey line; for Floor 3, the line shall pass through ‘ 2 ‘.

REFERENCE LINE 1 This reference line’ fixes a point on the line for any combination of values for basic stress and storeys

SPAN POINT The fourth line has a span point, through which all lines shall pass through for arriving at the thickness.

REFERENCE LINE 2 This reference line also fixes a point on the line for any combination of values for basic stress and storeys

PERCENTAGE OF OPENINGS

- The openings provided on the walls for windows, ventilators, doors, shelves, etc, are taken care of in the nomograms by this line.
- Window eight is taken as l.5 m for calculations.
- Openings which occupy up to 50 percent of the area of wall under consideration, come under the purview of the nomograms.

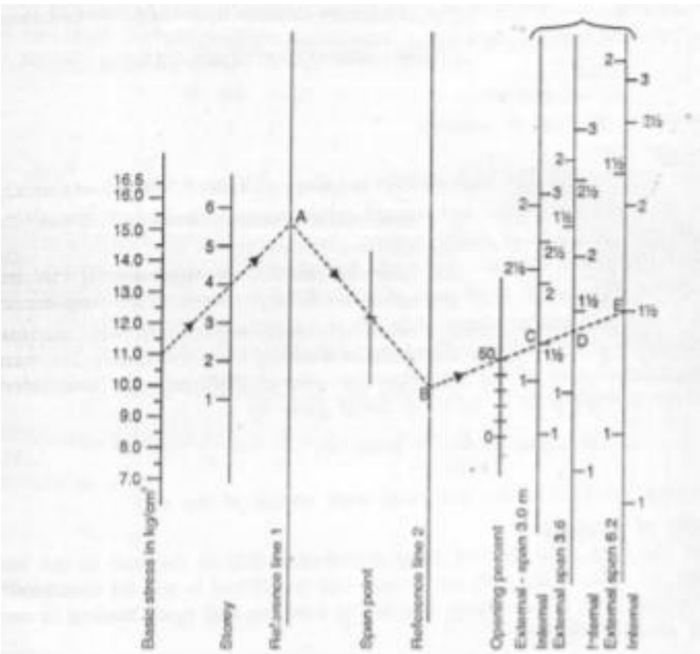
THICKNESS

- The last three lines in any nomogram give the thickness of brick wall for a particular loading and a storey height.
- The three sets of thicknesses are for three spans of the rooms, namely, 3.0, 3.6 and 4.2 m.
- Thicknesses are indicated on both sides of the lines.
- The bold markings on the left side of the lines give the thicknesses for external walls and the dotted markings on the right side of the lines give the thicknesses for internal walls.
- Internal walls are analyzed as walls having spans on either side.
- The numbers 1, 1.5, 2, etc, on these lines indicate the (number of) brick thickness; for example, 1 indicates 1 brick thick.

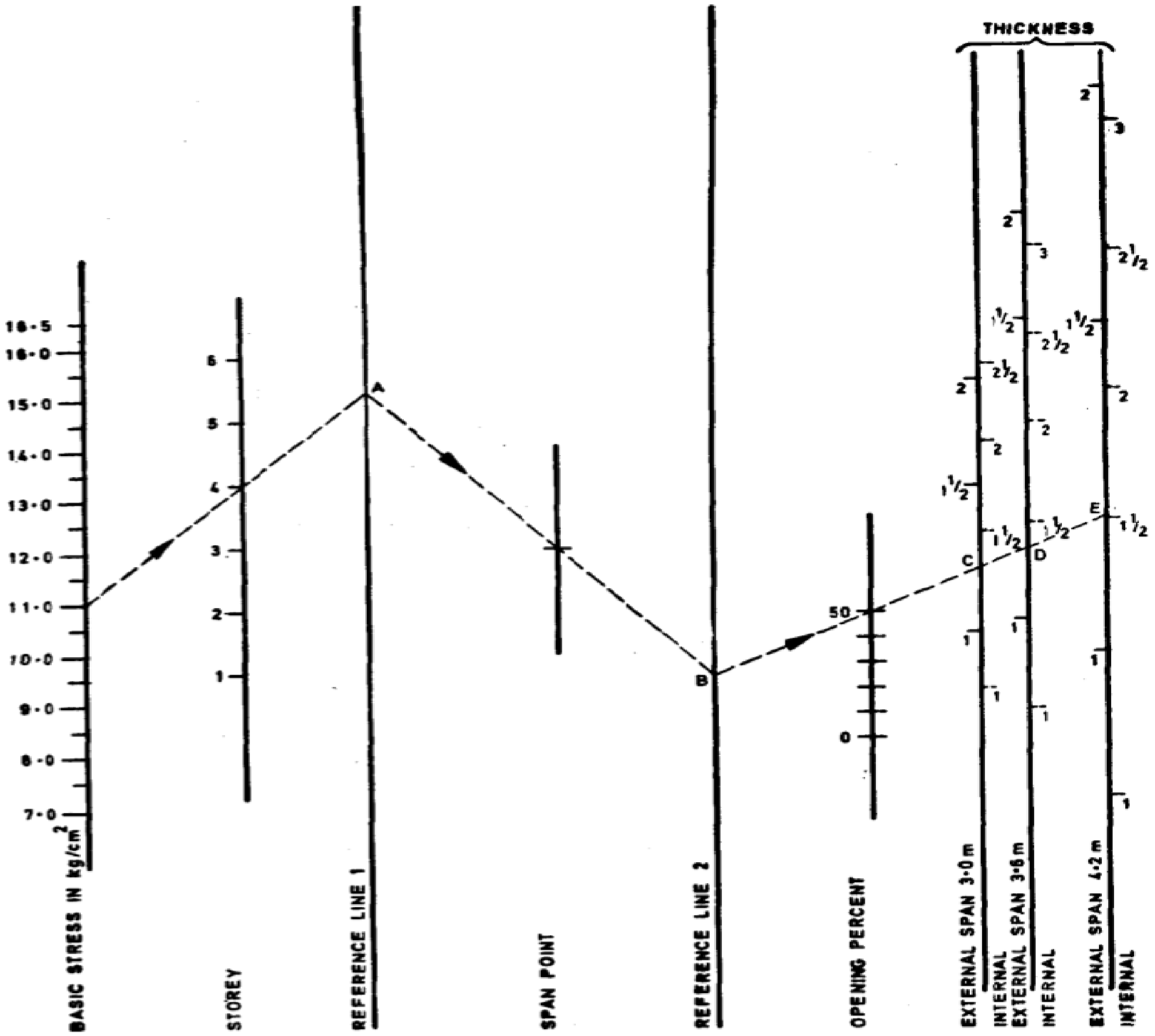
2.6.2 PROCEDURE FOR USE OF NOMOGRAMS

The Method Of use of nomograms for determining the thickness of brick wall has been illustrated by a dotted line. The following procedure may be adopted:

1. Choose the basic stress corresponding to the properties of masonry units and type of mortar to be used. For example, if we use bricks having crushing strength of 140 kg/cm^2 (14N/mm^2) and 1 : 1 : 6 cement-lime mortar, the basic compressive stress, found from Table 1 of SP 10, will be 11 kg/cm^2 (1.1 N/mm^2). This stress of 11 kg/cm^2 is marked on the first line of the nomogram.
2. If the building has 4 storey's, and the wall is to De designed at the ground level, the point of 11 kg/cm^2 is joined to storey 4 of the second line (story line), and extended to cut reference line No. 1 at point A.
3. Join A to the span point and prolong further to cut the reference line No. 2 in point B
4. Suppose the percent openings in the wall are 50. Join the Point B to 50 mark on the 6th line (opening line), and extend it further to cut the thickness lines in C, D and E.
5. The thickness of wall shall be the value of the dividing line which appears
6. From the nomogram, for the points of intersection C, D and E, the following thicknesses are obtained:



Point	Span (m)	Thickness (In Brick Thickness)	
		External	Internal
C	3	1.5	1.5
D	3.6	1.5	1.5
E	4.2	1.5	2



This nomogram is valid for the following conditions:

i) Buildings (residential):

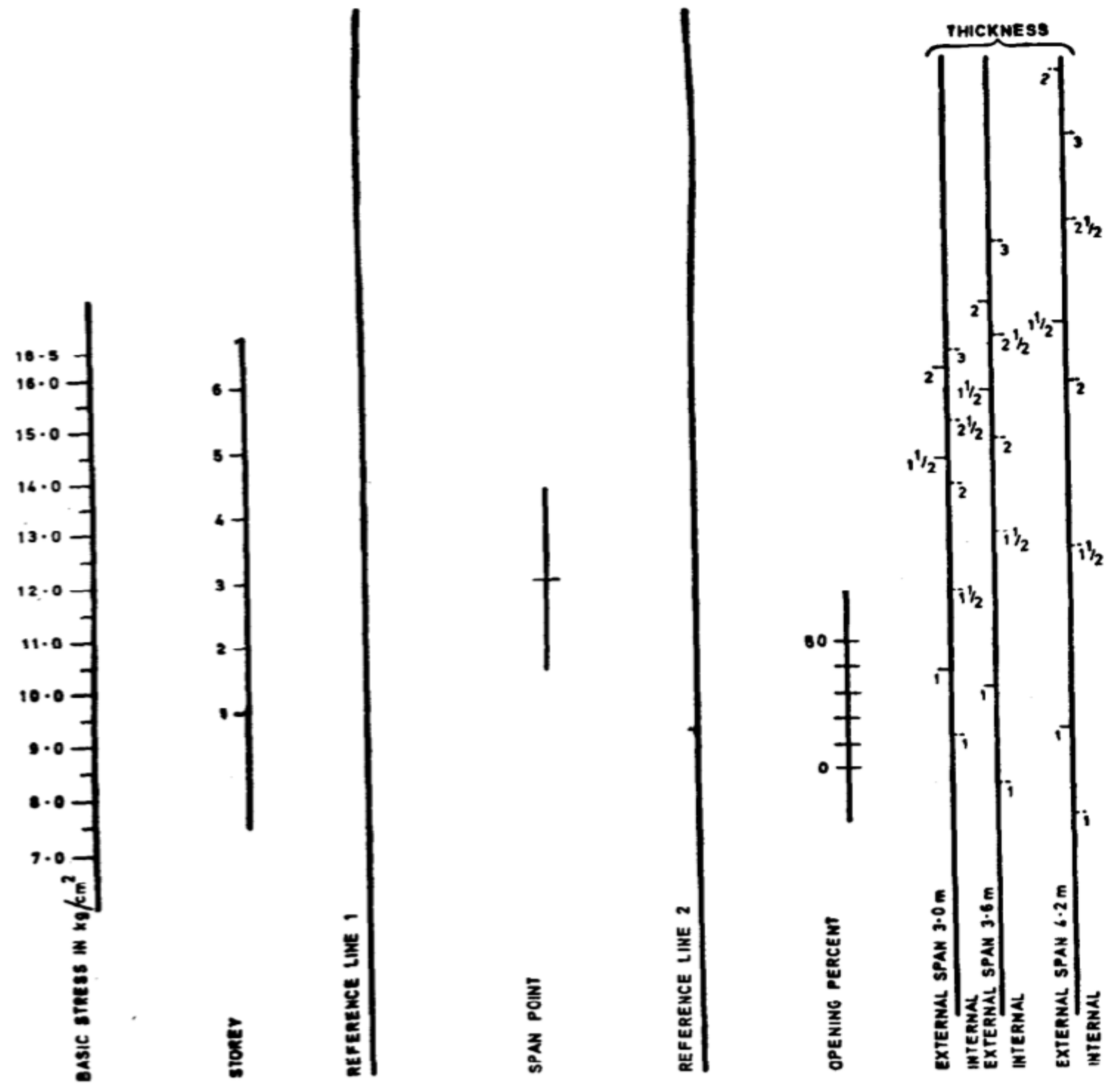
— Live loading	...	200 kg/m²
— Dead loading (assumed)	...	415 kg/m²

ii) Storey height ... 2·8 m

1A For Residential Buildings (Class 200 Loading) with 2·8 m Storey Height

FIG. 1 NOMOGRAMS FOR THICKNESS OF BRICK WALLS

SP : 10 - 1975



This nomogram is valid for the following conditions:

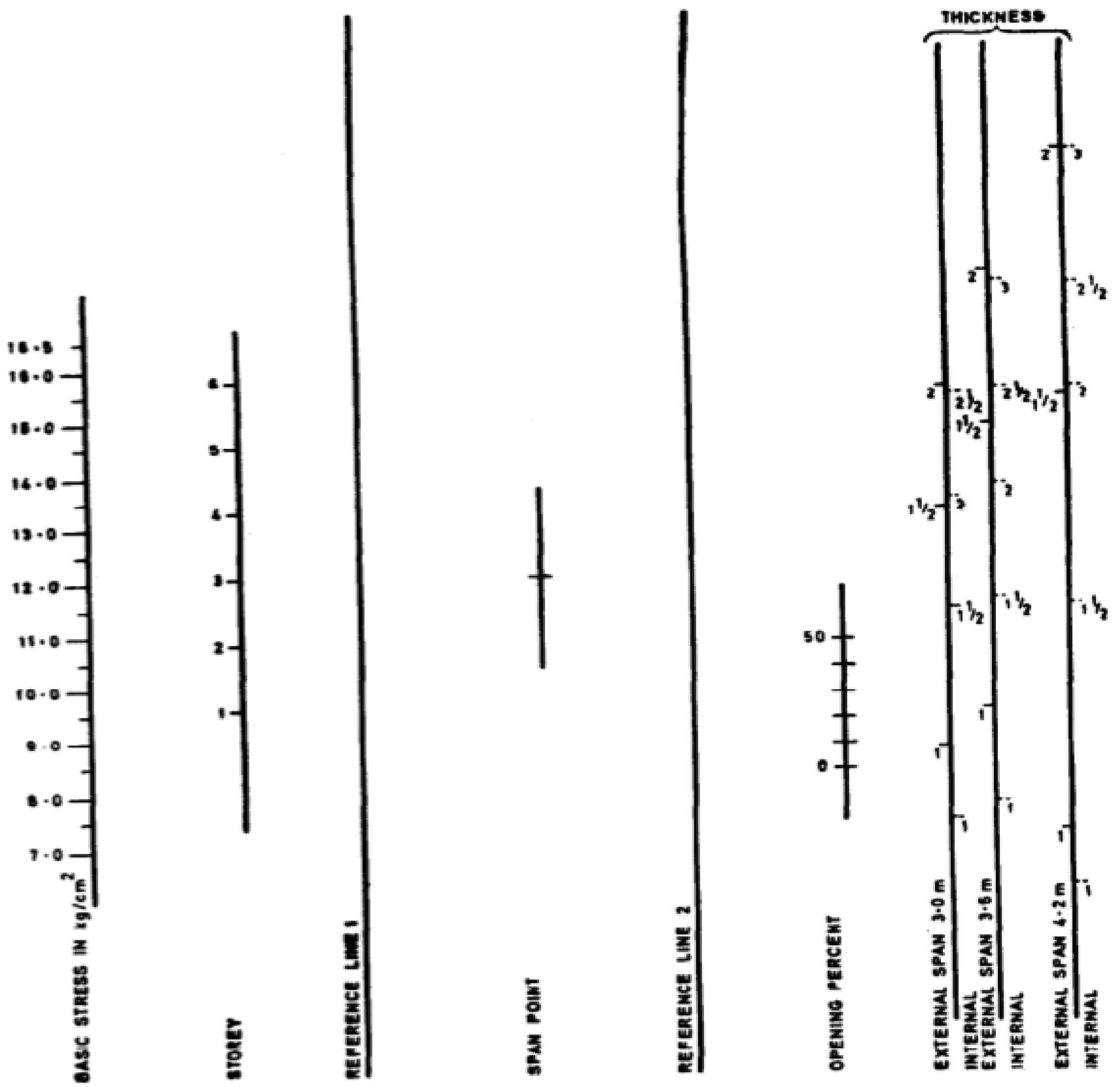
i) Buildings (residential):
— Live loading ... 200 kg/m²
— Dead loading (assumed) ... 415 kg/m²

ii) Storey height ... 3·2 m

1B For Residential Buildings (Class 200 Loading) with 3·2 m Storey Height

FIG. 1 NOMOGRAMS FOR THICKNESS OF BRICK WALLS

SP : 10 - 1975



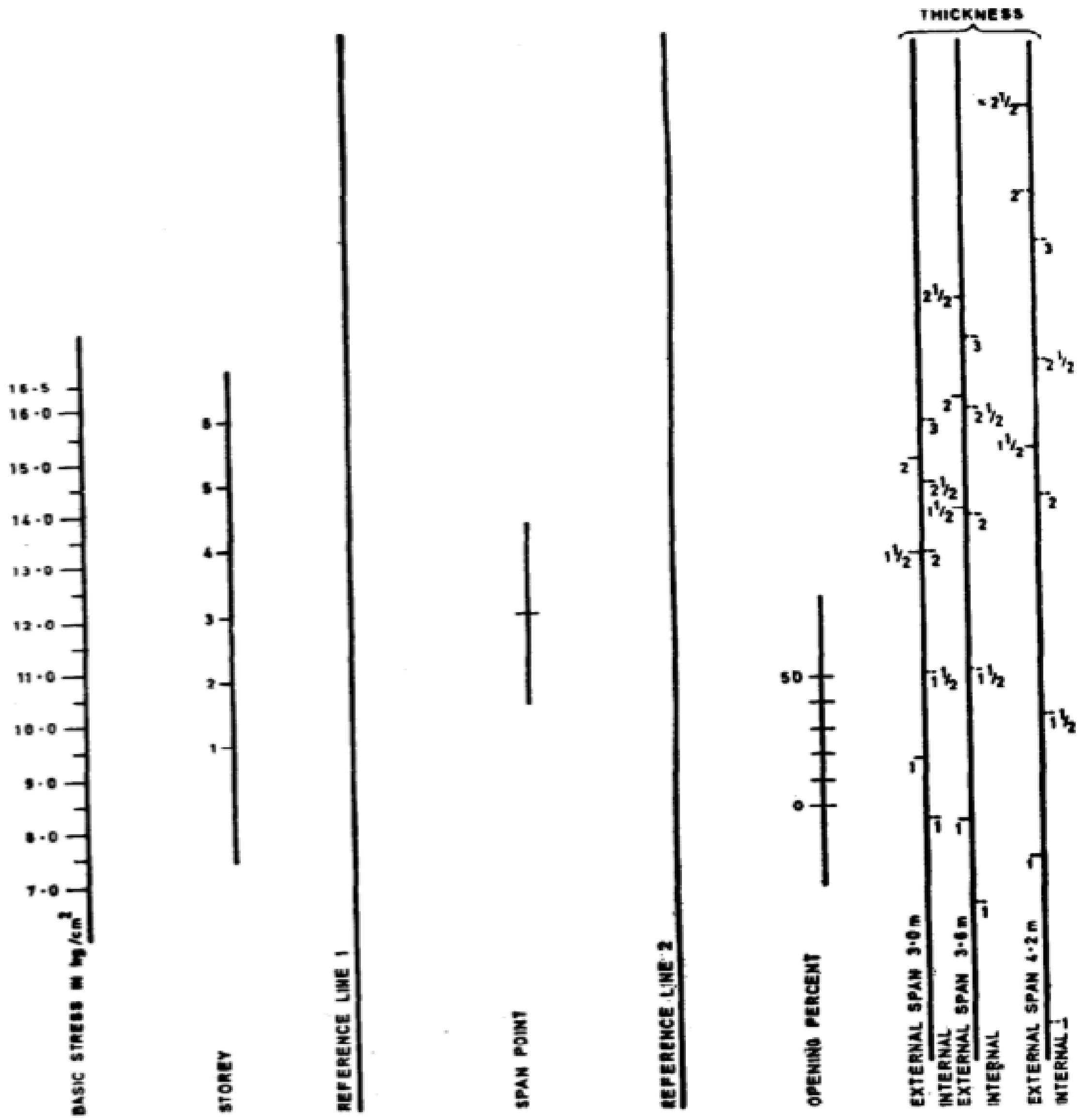
This nomogram is valid for the following conditions:

i) Buildings (office):
— Live loading ... 300 kg/m²
— Dead loading (assumed) ... 440 kg/m²

ii) Storey height ... 3·0 m

1C For Office Buildings (Class 300 Loading) with 3·0 m Storey Height

FIG. 1 NOMOGRAMS FOR THICKNESS OF BRICK WALLS



This nomogram is valid for the following conditions:

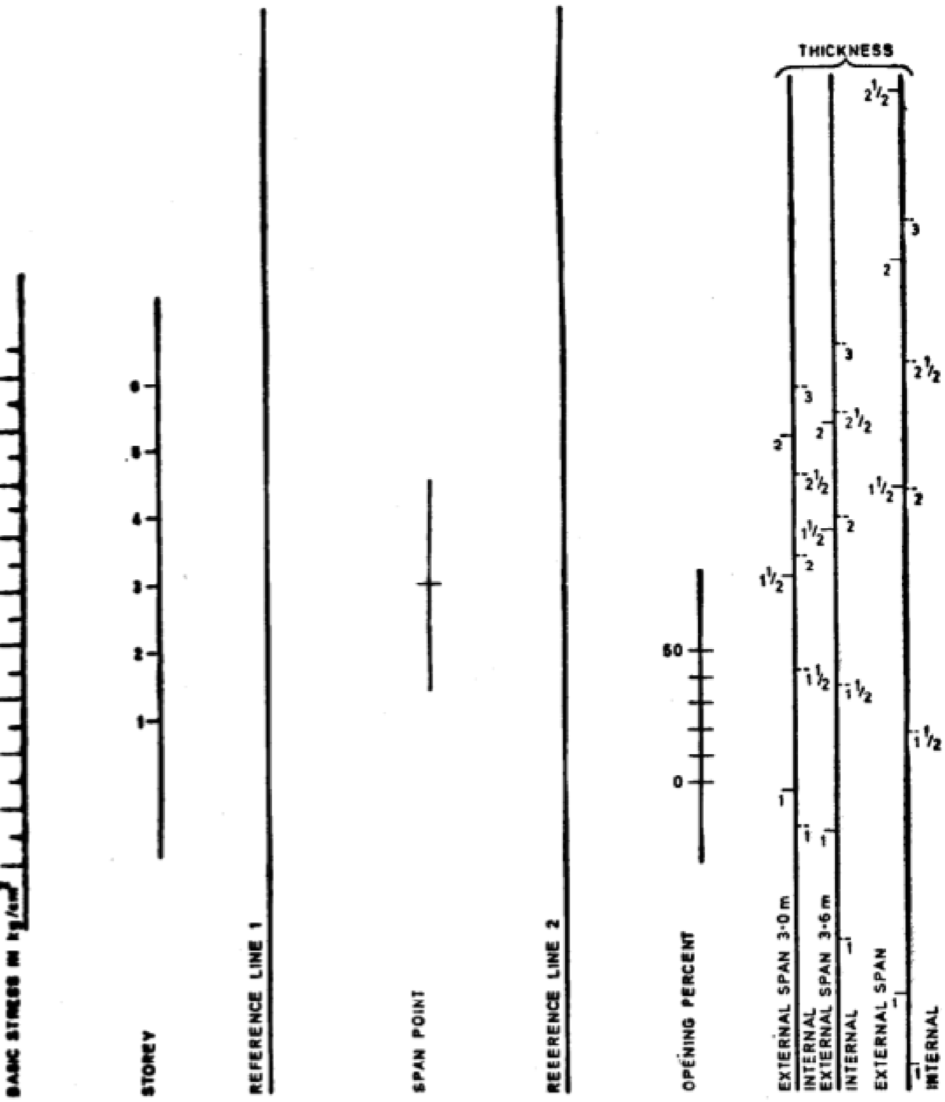
i) Buildings (office):
— Live loading ... 300 kg/m²
— Dead loading (assumed) ... 440 kg/m²

ii) Storey height ... 3·4 m

1D For Office Buildings (Class 300 Loading) with 3·4 m Storey Height

FIG. 1 NOMOGRAMS FOR THICKNESS OF BRICK WALLS

SP : 10 - 1975



This nomogram is valid for the following conditions:

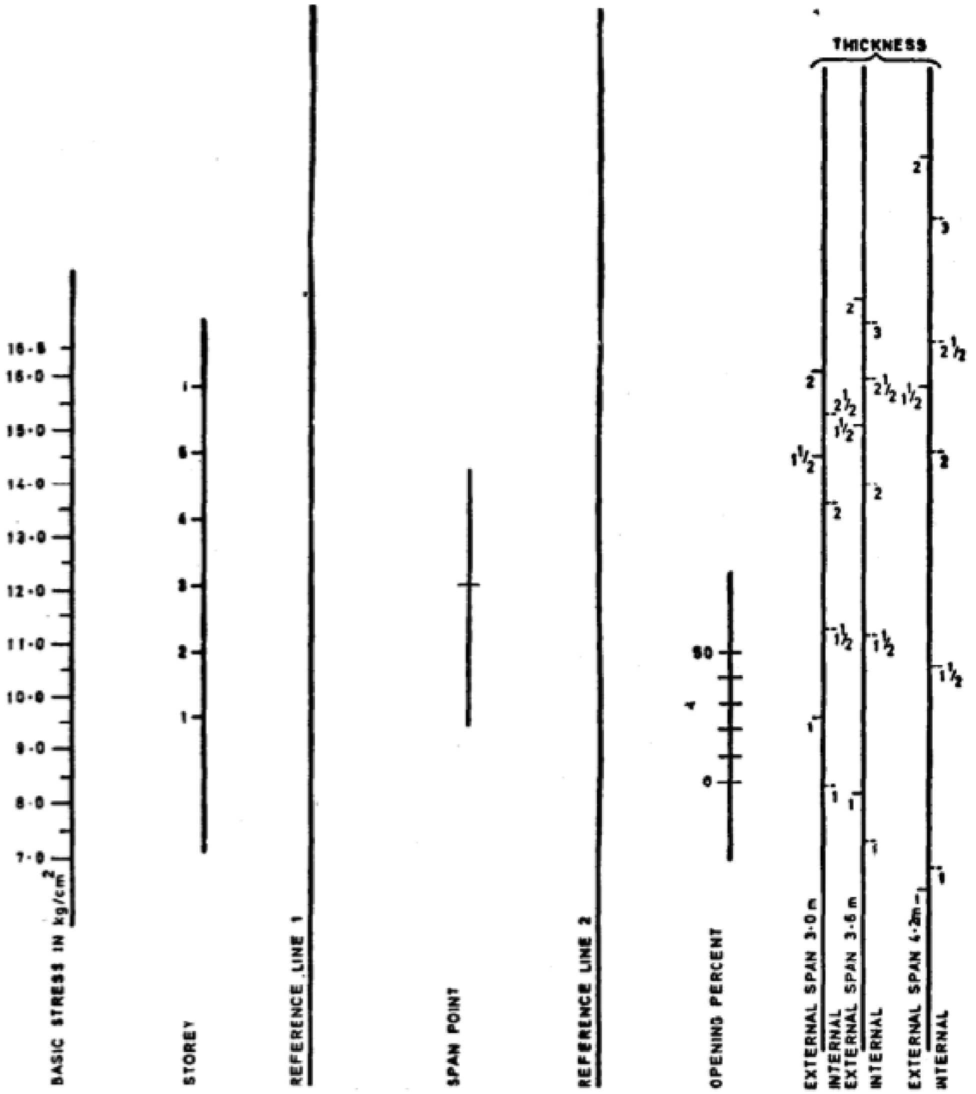
i) Buildings (office):

— Live loading	...	300 kg/m²
— Dead loading (assumed)	...	440 kg/m²

ii) Storey height ... 3·8 m

1E For Office Buildings (Class 300 Loading) with 3·8 m Storey Height

Fig. 1 Nomograms for Thickness of Brick Walls



This nomogram is valid for the following conditions:

i) Buildings (office):

— Live loading ... 400 kg/m²

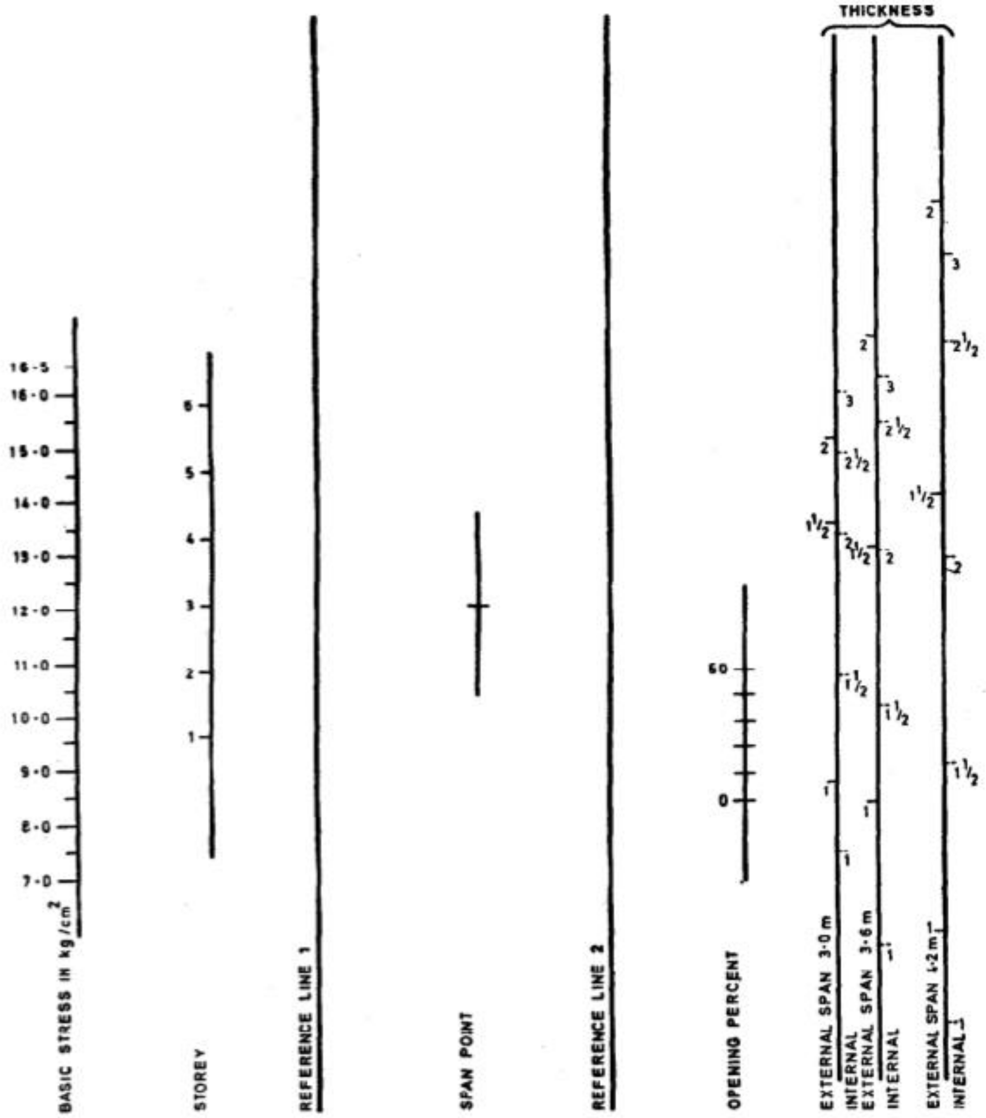
— Dead loading (assumed) ... 490 kg/m²

ii) Storey height ... 3·0 m

1F For Office Buildings (Class 400 Loading) with 3·0 m Storey Height

FIG. 1 NOMOGRAMS FOR THICKNESS OF BRICK WALLS

SP : 10 - 1975



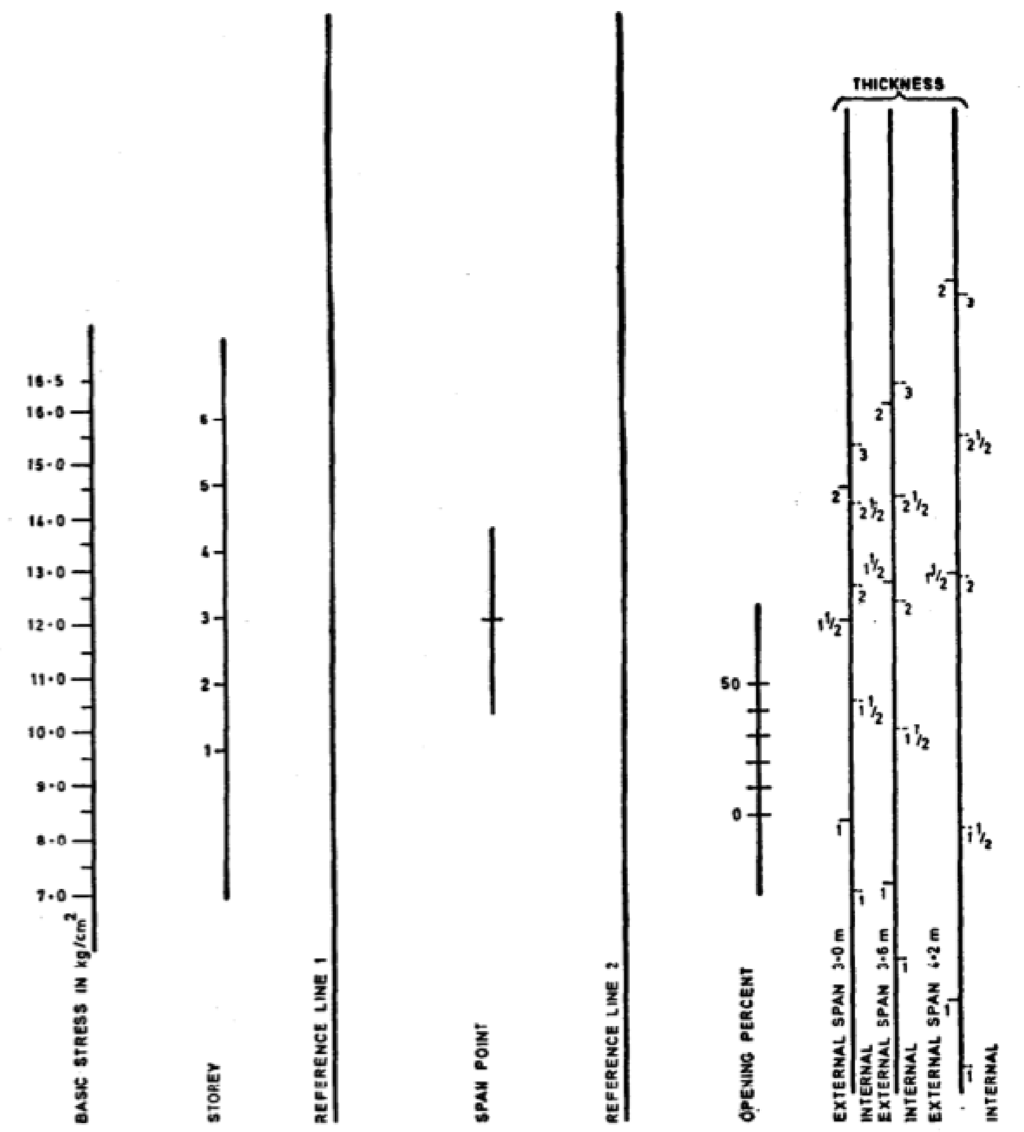
This nomogram is valid for the following conditions:

i) Buildings (office):
- — Live loading ... 400 kg/m²
- — Dead loading (assumed) ... 490 kg/m²

ii) Storey height ... 3·4 m

1G For Office Buildings (Class 400 Loading) with 3·4 m Storey Height

FIG. 1 NOMOGRAMS FOR THICKNESS OF BRICK WALLS



This nomogram is valid for the following conditions:

i) Buildings (office):
- — Live loading ... 400 kg/m²
- — Dead loading (assumed) .. 490 kg/m²

ii) Storey height ... 3·8 m

1H For Office Buildings (Class 400 Loading) with 3·8 m Storey Height

FIG. 1 NOMOGRAMS FOR THICKNESS OF BRICK WALLS

2.6.3 USES OF NOMOGRAMS

- It is graphical analog computation device
- The answer is approximate and useful
- Nomograms are used to check an answer obtained from an exact calculation method
- The unique advantage of nomograms is its visualizing of the relationship of involved parameters. This offers the possibility of playing around with new values and different assumptions and of optimizing solutions in an iterative process in line with actual requirements. A further advantage of nomogram s is that it doesn't matter which parameter is unknown. A solution may therefore be found from different angles.
- Using the same nomogram for a specific type of problem again and again allows you to develop a feeling for the optimal solution.
- Furthermore, nomograms prevent you from calculation mistakes, but require careful scale reading.
- Similarly, diagrams have the advantage of showing a specific point and its relation to other possible solutions.
- Diagrams as well as nomograms define the range of application and therefore provide guidelines as to the validity of assumptions made

2.6.4 LIMITATIONS OF NOMOGRAMS

- Nomograms are limited by precision with which physical markings can be drawn, reproduced, viewed and aligned
- Nomograms should not be used blindly, under no circumstances it exempts you to understand what you are doing.

UNIT 3

DESIGN OF TIMBER BEAMS

3.1 INTRODUCTION

Timber is one of the most useful and important material for constructions. Selecting timber is not an easy task, because timber has different types out of which selecting the right material is an important key. Timber is an expensive material to be incorporated in a building for different purpose therefore it should necessarily be strong, tough and durable. Timber doors or windows and etc. contribute a lot in the beautification and overall look of interiors. Timber is used in doors, windows, cabinet, cupboards, shelves, tables and railings etc. Timber is also popularly used in the form of plywood & raw wood. Products like ply blocks and ply boards. Heavy patterned doors and windows are made of solid wood/Timber to provide the strength, toughness and durability. Type of timber to be used for right purpose is important because if timber used in construction is of low quality then this may need replacement. While selecting timber one should consider its quality aspect as timber must be free from decay like rotten, fungi and termite.

3.2 TIMBER

Timber, being a natural product available in abundance in India, is used for the construction of doors, windows, roofs, partitions, beams, post, cupboards, shelves, etc. The strength characteristics of various types of timbers, generally used in India for engineering works. Generally, the trees from which timber is obtained are classified in two groups:

- Exogenous or outward growing trees, and
- Endogenous or inward growing trees.

The former is the class of trees which includes all the commercial timbers used for building purposes, and the latter class includes the trees, such as palms and bamboos, not suitable for engineering use and are found in the tropics.

3.2.1 HARD WOOD & SOFT WOOD

The exogenous trees are further divided into two main groups:

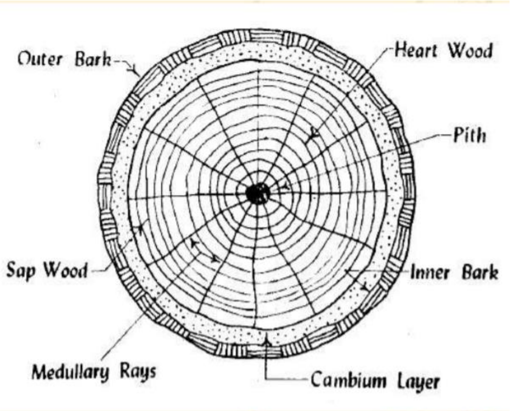
- Hard woods
- Soft woods.

One should not misunderstand this nomenclature used in classification (i.e., hard wood is not necessarily possessing more strength) because it is purely a botanical classification. It has been found that many hard woods are softer than certain soft woods.

- Hard woods have broad leaves, dark color, are generally dense and have narrow and well-defined annular rings. They are heavy, strong and hard. Hard are capable of resisting all stresses equally well.
- Soft woods have long and narrow pointed leaves and are characterized by distinct annular rings: have straight grains, more uniform texture and light color. Soft woods are very strong for direct pull but weak in resisting thrust or shear.

3.2.2 STRUCTURE OF A TIMBER

The trees which grow outwards about the center are called exogenous trees. Exogenous trees such as Sal, Babul, Teak, etc. are used for all engineering purpose. If trunk of tree is cut in transverse direction, the annual rings can be seen which are useful in predicting the age of tree. The cross – sectional parts of tree and their functions are described below (Parts from outer to inner).



BARK OR SKIN

It is the outermost portion of tree. The out bark protects the tree from high temperature and mechanical harm. The inner Bark covering the cambium layer protects cambium layer from any injury and also supplies food to those parts where it is needed most.

CAMBIUM LAYER

This layer of sap between inner bark and sapwood is known as cambium layer. This layer makes bark on the outer side and wood on inner side. It is an immature sap wood.

SAP WOOD

The outer annual rings between cambium layer and heart wood are known as sap wood. Its is usually light in color and weight as compared to heart wood. Sap (i.e., plant juice and moisture) actually is the food of tree which is made available to the tree from the ground through roots. Sap wood is not used for any engineering work because it contains large amount of moisture and is liable to quick decay.

MEDULLARY RAYS

These are thin radial fibres between cambium layer and pith of tree. The function of these rays is to hold together the annual rings of sapwood and heartwood. In addition to this, these rays store the food of the tree and distribute to different parts according to necessity.

HEART WOOD

The annual rings between sapwood and pith from the heartwood. The rings of heartwood are closer than the rings of sapwood. Heartwood is darker in color and is strong and durable. Actually, the sapwood gets converted to heartwood in due course of time, usually 10 to 30 years. Heartwood is dead portion of tree and does not take part in the growth of tree. Heartwood is used for all engineering works.

MEDULLARY SHEATH

It is layer between heartwood and the pith. It covers the pith from all around.

PITH

It is the innermost portion of the tree. Pith is very soft portion and varies in size and shape for different types of trees. It consists entirely of cellular tissues.

3.2.3 CHARACTERSICTICS OF GOOD TIMBER

APPEARANCE	A freshly cut surface of timber should exhibit hard and of shining appearance.
COLOR	A colour should preferably be dark
DEFECTS	A good timber should be free from series defects such as knots, flaws, shakes etc.,
DURABILITY	A good timber should be durable and capable of resisting the action of fungi, insects, chemicals, physical agencies, and mechanical agencies. Class I : Average life 120 months and over Class II : Average life 60 to 119 months Class III : Average life 59 months and below
ELASTICITY	The timber returns to its original shape when load causing its deformation is removed.

FIRE RESISTANCE	A dense wood offers good resistance to fire so that it does not easily ignite. It helps in fire protection of buildings.
MECHANICAL WEAR	A good timber should not deteriorate easily due to mechanical wear or abrasion.
SHAPE	A good timber should be capable of retaining its shape during conversion or seasoning
SMELL	A good timber should have sweet smell. Unpleasant smell indicates decayed timber Sound: A good timber should give a clear ringing sound when struck.
STRENGTH	A good timber should be sufficiently strong for working as structural member such as joist, beam, rafter etc.
TOUGHNESS	A good timber should be tough (i.e.) capable of offering resistance to shocks due to vibration.
STRUCTURE	The structure should be uniform
WATER PERMEABILITY	A good timber should have low water permeability, which is measured by the quantity of water filtered through unit surface area of specimen of wood
WEIGHT	The timber with heavy weight is considered to be sound and strong
WORKING CONDITIONS	Timber should be easily workable. It should not clog the teeth of saw
WEATHER RESISTANCE	A good timber should posses adequate resistance against weathering effects such as alternate drying and wetting, alternate heating and cooling because of temperature variations, wind effects, etc.,
HARDNESS	A good timber should be hard.
WORKABILITY	The timber should be easily workable and should not clog the teeth of saw. It should also be capable of being easily planed or made smooth.
OTHER PROPERTIES	• Timber should have sufficient weight. A timber with heavy weight is considered to be sound and strong • The structure of timber should be uniform, hard and compact • A timber should have sufficient hardness i.e, resistance against penetration • A timber should have favourable physical characteristic such as dark colour, straight fibres, shining appearance, free from straight fibres, shining appearance, free from defects, sweet smell, good sound when struck etc.,

3.2.4 DEFECTS OF TIMBER

The structural timber to be used for building purpose should be free from all defects, viz., :

- Due to insects such as beetles, marine borers, termites, etc.,
- Defects developed during conversion into commercial forms such as chip marks, diagonal marks, depression on structure, etc.
- Due to fungi,
- Due to natural forces such as knots, rind galls, shakes, upsets, cracks, et., and
- Defects developed during seasoning process such as splitting, warping, twisting, honey combing, radial shakes, etc.

Most common defects in timber are:

HEART SHAKES

These are splits occurring in the centre of the tree and running from the pith (inner most part) towards the sap wood from the medullary (vascular tissues) rays. In some timbers, these splits are hardly visible and, in some timbers, these are quite permanent. Heart shakes are caused due to shrinkage of interior parts due to age. A heart shake straight across the trunk is not a serious defect.

STAR SHAKES

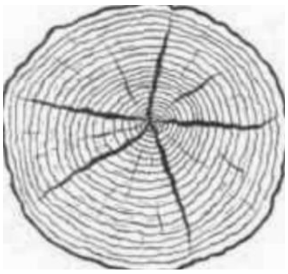
These are splits which radiate from the centre of the timber or from the bark (outer side), running in the planes of medullary rays. These occur due to severe frost or scorching heat of the sun.

CUP SHAKES

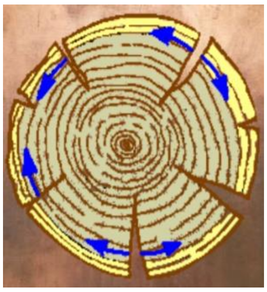
These are curved splits which separate the whole or part of one annual ring from another. These are caused due to the unequal growth of the timber.

RADIAL SHAKES

These are similar to the star shakes and occur in felled timber when exposed to the sun during seasoning. Radial shakes are generally irregular, fine and numerous. In this many splits are appeared.

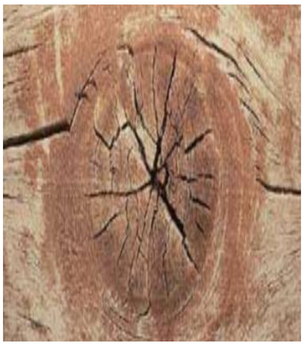


Heart Shake



Star Shake

Cup Shake



Radial Shake

RIND GALLS

These are typical enlarged swellings and occur due to branches cut-off.

WIND CRACKS

These are shakes or splits on the sides of a bark of timber due to shrinkage of exterior surface exposed to atmospheric influence

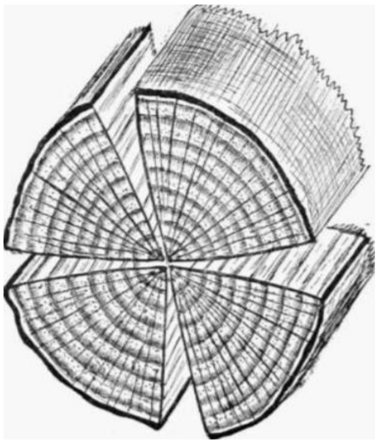
KNOTS

These are the roots of the small branches of the tree. These are not harmful.

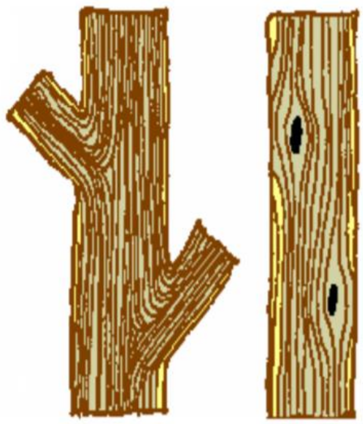
DEAD WOOD

It is the deficient in strength and weight and is the result of trees being felled after maturity.

Rind Galls



Wind Cracks



Dead Wood

3.2.5 DISEASES OF TIMBER

The Disintegration and turning to powder of timber Tissues is known as Decay or Disease of Timber. When the timber, without proper seasoning, is placed in a position where there is no circulation of air, The disintegration of wood tissues is caused by fungi or other micro – organism. Dampness, alternate wet and dry conditions and old age accelerate decay of timber.

The common diseases of timber are,

- Dry rot
- Wet rot

DRY ROT

The turning of timber tissues to almost a dry powder by fungi is called dry rot. There are many kind of fungi (means mushroom or any kind of spongy plant)which gain admittance into the Wood by means of pores, which breed in wood cells And spread minute thread like growth.

Favorable Conditions of Dry Rot

Sufficient temperature, Dampness, lack of free air and adequate food material. The fungus feed upon the wood and eats the wood Tissues, thus penetrating the wood fibres in all directions. The wood is reducing to almost a dry powdery form by eating away the interior fibred of wood. The wood becomes soft like spongy mass usually grey in colour. Sufficient temperature, Dampness , lack of free air and adequate food material.

Prevention of Dry rots

The timber which is well seasoned and free from sap, should be used because mostly sapwood is attacked by fungus. It should be used at a place where there is free circulation and access of fresh air.

Remedy of Dry rots

The infected part of timber should be burnt or cut away & remaining portion should be painted with a solution of copper sulphate or with any other suitable preservative. The wood cells which form the food of the fungi, should be impregnated with poison to control further growth of fungi. The high temperature of seasoning kiln, will also Help in killing the fungi & expelling the moisture.

WET ROT

The disintegration of tissues of the timber due to alternate wetting & drying is called wet rot. Common found in a Living tree. The attack takes place through wounds in the bark by the access of water you can say high % of moisture. The decomposition of the effective part results in converting the wood fibers into a greyish brown powder.

Prevention of Wet rots

All timbers for exterior or underground work should be first properly seasoned and then coated with tar or some paint to keep out the dampness.

Remedy of Wet rots

for treating wet rot is by using a suitable preservative

3.2.6 SEASONING OF TIMBER

As fresh timber which is obtained from trees contains about 30 to 40 % sap or moisture. This sap is very harmful for the life of a timber. Therefore, it is necessary to remove that sap by applying some special methods. All those methods which are used for removing the sap from timber are collectively termed as seasoning of timber. *"Seasoning is the process of drying out the timber to a moisture content approximately equal to average humidity of situation where it is to be used"*

ADVANTAGES OF SEASONED TIMBER

- It has reduced weight
- It is strong and durable
- It has resistance to decay or rot
- It takes high polish
- It is easier to work
- Its life is more.

TYPES OF TIMBER SEASONING

The main types of timber seasoning are as under.

1. Natural Seasoning
2. Artificial Seasoning

NATURAL SEASONING

In the air seasoning or natural seasoning or natural drying, seasoning of timber, timber is dried by direct action of air, wind and sun. In this method, the timber logs are arranged one over the other, keeping some space or distance between them for air circulation of fresh air. Generally, this type of seasoning requires few months to over a year, this is very slow process. Generally, for every 25cm thickness of sawn timber, soft wood takes about 2 to 3 months and hard wood about 12 months.

ARTIFICIAL SEASONING

The various methods of artificial seasoning are adopted against natural seasoning because of the following reasons.

- The process of drying is controlled and hence not liable to be attacked by insects and fungi
- Period of seasoning can be reduced depending upon the need
- Since the desired moisture content in timber can be attained, so it minimises the defects such as shrinkage, cracking, warping etc.,
- Drying of different surfaces is even and uniform due to better control of air, humidity and temperature.

The different methods of artificial seasoning are,

- Water seasoning
- Seasoning by boiling / steaming
- Kiln or Hot air seasoning
- Chemical / Salt seasoning
- Electric seasoning
- Smoke drying
- Charring or Scorching

WATER SEASONING

In water seasoning, timber logs are kept immersed whole in the flowing water. Sap present in timber is washed away. After that log are taken out from water and are kept in open air, so water present in timber would be dried by air. The time required for this is seasoning is 2-4 weeks. Although it is a quick process but reduces the elasticity & durability of timber and makes it weak & brittle.

SEASONING BY BOILING OR STEAMING

In this method timber is first immersed in water and then water is boiled for 3 to 4 hours. In this timber is exposed to action of steam spray. The timber is thus dried out slowly and seasoned. This is very quick method of seasoning and reduces the shrinkage. However, this is expensive method and affects the strength and elasticity of timber.

KILN OR HOT AIR SEASONING

In this method, timber can be seasoned to any moisture content and hence commonly adopted for rapid hardening of timber on a large scale. In this process the timber is stacked inside the chamber and hot air is forced for free circulation for about three days. The sap and moisture content are out to the desired extent. It is carried either in stationary kilns or in progressive kilns. In kiln seasoning timber is placed in a chamber with some special heating arrangement. In this process one thing should be kept in mind that heating system should be under control, otherwise timber will be crack or wrap. The time required for this seasoning is 3 to 12 days. This is quick process. This method of seasoning gives a well-seasoned timber as it controls three important conditions namely, air circulation , relative humidity and temperature.

CHEMICAL OR SALT SEASONING

In chemical seasoning carbon dioxide, ammonium carbonate or urea are used as agents for seasoning, those are applied in dry state, the inter surface of timber dries first than outer side. This ensures uniform seasoning. The timber then taken out and seasoned by ordinary way i.e., by air seasoning. The time required for this seasoning is 30 to 40 days. There is less formation of external cracks.

ELECTRIC SEASONING

Green timbers offer less resistance to flow of current as compared to dry timber. Therefore, high frequency alternating currents which produce heat are used to dry out the timber. In this method electric current is passed through the timber logs. The time required for this seasoning is 05 to 08 hours. This is most rapid method of seasoning but being very costly cannot be used for seasoning of timber on a large scale.

SMOKE DRYING

In this, timber is dried out over a fire straw or twig. Care should be taken in applying heat gradually to prevent splitting. Timber seasoned by this method is hardened, more durable and proof against worm attacks. This method is generally used for bending planks in boat building.

CHARRING OR SCORCHING

In this process the ends of piles or posts are burnt to charcoal in order to remove moisture content. This method is used for piles, posts and other members that are to be kept under water or ground. However, this produces dry-rot in green timber.

3.2.7 PRESERVATION OF TIMBER

It is the process of protecting or preserving the timber structures from the attack of destroying agencies such as moisture, dry-rot, internal decay, fungi, insects, etc., preservation also ensures increased life of timber and better durability.

PRINCIPLE OF PRESERVATION

The basic principle employed for preservation of timber is to poison the food of fungi in the form of preservatives. The success of preservative treatment depends on proper choice of preservatives and method of its application.

REQUIREMENTS OF GOOD PRESERVATIVE

- Timber is a cheaper material and hence preservative should also be cheap so as to limit the overall cost of the member
- It should be easily available and capable of being applied by non-skilled or semi-skilled person
- It should be quite efficient in killing fungi and insects but should be safe and harmless for persons and animals
- It should offer high resistance to fire, moisture and dampness
- It should be pleasing in appearance after treatment, but should be odourless and colourless
- It should have the capability of covering a large area with a smaller quantity
- It should have high penetrating power, stability and durability
- It should not be affected by heat and light
- It should not be easily washed away by water
- It should not corrode or react with the building materials which come into contact with the timber.

TYPES OF PRESERVATIVES

The following are the types of preservatives commonly used:

- Oil preservatives : Creosote oil, Oil paints, Solignum paints, Coal tar, etc.,
- Water soluble preservatives : Zinc chloride, Boric acid, Sodium Fluoride etc.,

OIL PRESERVATIVES

They are generally employed for out-door and wet exposure conditions. Although they possess high toxicity and non-corrosive qualities but they offer painting difficulties. They are adopted for treatment of members like posts, poles, piles etc. since they are difficult to handle and give unpleasant odour.

WATER SOLUBLE PRESERVATIVES

These preservatives are commonly adopted for interior wood work since they are washed away by rain water. They are colourless, odourless, and involve very little fire hazard. Timber treated with such preservatives can be painted or varnished easily.

METHODS OF PRESERVATION

The following are the methods of preservation for treating wood:

- Charrying, Tarrying and Creosoting
- Surface Application: Brushing, Spraying and Dipping
- Soaking treatment
- Hot and Cold process
- Pressure process
- ASCU Treatment

CHARRYING:

It is the process of preserving timber without using preservative. In this process, timber to be preserved is kept wet for half an hour and then burnt to charcoal over wood fire. Finally, it is cooled and quenched with water. This method is used for posts, piles, etc., where timber is used in ground.

TARRYING:

It is the process of coating the timber with coal tar while hot. Door and window frames, piles, etc., built into walls and ground are generally tarred.

CREOSOTING:

Creosote oil is obtained by distillation of coal tar. The seasoned timber is kept in an air tight chamber and air is exhausted. Then creosote oil is pumped into the chamber at a pressure of 0.8 to 1.0 N/mm^2 at a temperature of 50°C. After 1 to 2 hours timber is taken out of the chamber.

BRUSHING:

In this method, hot oil preservative solution is applied liberally in several coats (Usually 2 coats) on the timber surface by good quality brushes.

SPRAYING:

In this method, solution of preservative is filled in a spraying pistol and then applied on timber surface under pressure. This is more effective and superior to brushing.

DIPPING:

In this method, timber to be treated is dipped or soaked for a short period in preservative solution. This gives better penetration to brushing or spraying. Depth of penetration depends upon the type of timber.

SOAKING TREATMENT:

In this process, timber is submerged in the preservative solution for long time until the required absorption is obtained.

HOT AND COLD PROCESS:

This process is considered to be most efficient non-pressure treatment process. In this process, timber is stacked in the tank and cold preservative, usually creosote, is then run into tank till the timber is completely submerged. The preservative is then heated to about 85^0C to 95^0C and maintained at this temperature for some time. The tank is then allowed to cool down gradually while the timber still submerged in tank. During this alternate process of heating and cooling, the air in the timber first expands and contracts which creates a partial vacuum. This partial vacuum sucks the preservative into the timber. This method is recommended for treating sap wood and easily treatable heart wood.

PRESSURE PROCESS / VACUUM PROCESS:

In this method, the preservative is injected under pressure into the timber. This is the most effective method of treating timber with preservative. This method is sub divided into 2 types:

- **Full cell or Bethel process** – In this process, the timber is placed in an airtight impregnating chamber from which air is pumped out. The preservative, generally creosote oil under a pressure varying from 7 to 12.5 kg/cm^2 and at a temperature of about 48^0C, is then forced into the chamber. This pressure is maintained till the desired absorption is obtained. By this pressure, the preservative gets injected in the timber. Finally, the pressure is released and vacuum again created to withdraw the excess of preservative. This method doubles the life of timber treated for and also protects it from dry rot and white ants. This process is usually adopted in railway sleepers, pile, poles, etc.,
- **Empty cell or Rueping process** – this process is comparatively cheaper to full cell process, as it requires much less preservative to give the desired absorption

ASCU PROCESS:

This preservative is developed by the Forest Research Institute, Dehradun. It consists of 1 part by weight of hydrated arsenic pent-oxide (As_2O_5, 2 H_2O), 3 parts by weight of copper sulphate ($CuSO_4 \cdot 5$ H_2O) and 4 parts by weight of potassium dichromate ($K_2Cr_2O_7$) or sodium dichromate ($Na_2Cr_2O_7 \cdot 2$ H_2O). This preservative is available in powder form. By mixing six parts of this powder with 100 parts of water, the solution is prepared. The solution is then sprayed over the surface of timber. This treatment prevents attack from termites. The surface may be painted to get desired appearance.

3.3 GRADING OF TIMBERS

The quality of sawn timber varies widely depending on the species, where and how it was grown and the age at which the tree was felled. Thus, quality of wood is determined primarily by the density of wood, and strongly influenced by the size and distribution of knots and other defects.

Grading separates the available material into groups so that marketing is rationalised and selection for a particular application is simplified. Since grading rules vary between soft wood and hard woods, these will be treated separately.

3.3.1 SYATEMS OF GRADING [IS 6534-1971]

All grading specifications shall be clearly distinguished between,

- Structural or stress grading
- Commercial or utility grading

STRUCTURAL GRADING

It is also known as stress grading. However, there is a small distinction between the two. Structural grading refers to the principle by which the material is graded on the basis of visible defects which have known effects on the strength properties of the material.

Stress grading refers to the principle by which the material is graded by consideration of maximum principle stresses to which it can be subjected.

Structural grading is further divided as:

- Grading based on known effects of defects and estimating accumulative value.
- Machine grading.

COMMERCIAL GRADING

It is also known as yard grading or utility grading refers to the principle by which the material is graded by consideration of usefulness of the material and price factors.

Commercial grading is further divided in the following classes:

- Grade A
- Grade B
- Grade C
- Grade D

GRADE A

It is based purely, and sometimes arbitrarily, on dimensions and general appearance. The dimensions of lengths and girths for logs, or lengths, widths and thicknesses of converted material are measured according to specified methods. This system is prevalent in Kerala and Mysore. Under these classifications, teak is placed in four grades with two sub-classes in each grade. In the case of other hardwoods, there are similarly four grades in Mysore (Coorg) but the dimensions are fixed separately for each of the species. In Kerala, there seem to be only two grades of hardwoods.

GRADE B

It is based on the best ultimate use of logs or converted material. Such a system is mostly prevalent in Andhra Pradesh and some parts of Tamil Nadu, and seems to be one of the quickest systems of grading and marking. The logs are classified into grades on the best use possible as for beams, planks, scantlings, etc, and each grade is further divided into 'A', 'B' and 'C' classes to indicate occurrence of defects. Only two lengths are recognized; 'long (that is, 5 m and above) and 'short' (that is, under 5 m). Each log is thus quickly stamped with the first letter of the grade classification, the sub-class, and 'L' or 'S' for 'long' and 'short', for example, BAL and PBS indicate, respectively, 'beam, A-class, long' and 'planks, B-class, short'. Sometimes another letter is also added to indicate the species, for example, 'T' for teak.

GRADE C

This classification is based on qualitative evaluation of defects and rough estimate of out-turn of utilizable material. It is prevalent in Madhya Pradesh.

GRADE D

It is based purely on evaluation of 'units of defects' and fixing the -number of units permissible for a standard volume in each grade. Such practices are common in the Bombay region; sometimes an estimated out-turn is also indicated in each grade. In general three grades are distinguished for various categories of logs and sawn timber. Sizes and other dimensions are also fixed in a few cases, separately for different species and different depots in the same state. This system is being increasingly adopted in the specifications of Indian Standards Institution, and in international grading specifications. This system has a distinct advantage of evaluating cumulative effect of defects in a particular grade.

3.3.2 GRADING OF CUT SIZES OF MEMBERS [IS 1331-1971]

Cut size of timber shall be graded after seasoning at a moisture content not less than 12%. Cut sizes shall be graded separately according to end uses, namely, structural and non-structural purposes.

GRADING FOR STRUCTURAL USE

Based on permissible and prohibited defects, the cut sizes of timber for structural use shall be of three grades,

- Grade 1
- Grade 2
- Grade 3

GRADE 1 : Estimated effect in reduction of the basic strength of timber is not more than 12.5%

GRADE 2 : Estimated effect in reduction of the basic strength of timber is not more than 25%

GRADE 1 : Estimated effect in reduction of the basic strength of timber is not more than 37.5%

GRADING FOR NON-STRUCTURAL USE

Based on permissible and prohibited defects, the cut sizes of timber for structural use shall be of two grades

- Grade 1
- Grade 2

Defects in timber			
Structural Use		Non- Structural Use	
Prohibited Defects	Permissible defects	Prohibited Defects	Permissible defects
Loose Grains, Splits, Compressive Wood in Coniferous Timber, Heart Wood Rot, Sap Rot, Warp, Worm Holes Made by Powder Post Beetles And Pitch Pockets	Wane, Worm Holes, Slope Of Grain, Live Knots, Checks And Shakes	Heart Wood Rot, Sap Rot, Brashness, Shakes, Insect Attack	Slope Of Grain, Knots, End Split, Pitch Pockets, Checks, Depth, Bow, Warp, Spring, Cracks, Twist, Centre Heart, Cup,Wane

3.4 PERMISSIBLE STRESSES

Fundamental stress values of different species of timber are determined on small specimen in accordance with standard practice laid in IS 1708 (Parts 1 to 18) : 1986. In these values are then applied appropriate reduction factors given in the relevant table of IS 3629 : 1986 to obtain the permissible stresses.

The permissible stresses for Groups A, B and C for different locations of use and applicable to Grade-I of structural timbers shall be as given in Table 1; and the corresponding minimum permissible stress limits shall be as given in Table 3, provided that the following conditions are met:

a. The timber should be of high moderate durability and be given suitable treatment where necessary
b. Timber of low durability shall be used after proper preservative treatment in accordance with IS 401 : 1982
c. The loads should be of continuous and permanent type.

For permissible stresses (excepting E) of other grades of timber, values given in Table 1 and Table 3 shall be multiplied by the following factors, provided that the conditions laid down in the above are satisfied:

- For select Grade Timber : 1.16
- For Grade II Timber : 0.84

When low durability timbers are to be used on outside location, the permissible stresses for all grades of timber, arrived above shall be multiplied by 0.80.

Minimum Permissible Stress Limits in Three Groups of Structural Timbers (For Grade I Material)

S.NO	Strength Character	Location of Use	Group A	Group B	Group C
1	Bending and Tension along grain	Inside	18	12	8.5
2	Shear – Horizontal	All locations	1.05	0.64	0.49
3	Shear -Along grain	All locations	1.5	0.91	0.70
4	Compression parallel to grain	Inside	11.7	7.8	4.9
5	Compression perpendicular to grain	Inside	4	2.5	1.1
6	Modulus of Elasticity	All location and grade	12.6	9.8	5.6

1. The values of horizontal shear to be used only for beams. In all other cases shear along grain to be used
2. For working stresses for other location of use, that is, outside and wet, generally factors of 5/6 and 2/3 are applied

3.4.1 MODIFICATION FACTORS FOR PERMISSIBLE STRESSES

DUE TO CHANGE IN SLOPE OF GRAIN

When the timber has not been graded and has major defects such as slope of the grain, knots and checks or shakes (but not beyond permissible values), the permissible stresses given in Table 1 of IS 883-1994 shall be multiplied by the modification factor K_1 for different slopes of grain as given in table.

Modification factor K_1 to allow for change in Slope of Grain		
Slope	K_1	
	Strength of Beams, Joists and Ties	Strength of Posts or Columns
1 in 10	0.80	0.74
1 in 12	0.90	0.82
1 in 14	0.98	0.87
1 in 15 and Flatter	1.00	1.00

DUE TO DURATION OF THE LOAD

For different durations of design load, the permissible stresses given in Table 1 of IS 883 shall be multiplied by the modification factor K_2 given in the table. The factor K_2 is applicable to modulus of elasticity when used to design timber columns, otherwise they do not apply thereto. If there are several durations of loads to be considered, the modification factor shall be absed on the shortedt duration load in the combination, that is, the one yielding the largest increase in the permissible stresses, provided the designed section is found adequate for a combination of the other longer duration loads.

Modification factor K_2 for Change in Duration of Loading		
S.NO	Duration of Loading	Modification facto, K_2
1	Continuous (normal)	1.00
2	Two months	1.15
3	Seven days	1.25
4	Wind and Earthquake	1.33
5	Instantaneous or Impact	2.00

3.5 DESIGN CONSIDERATIONS FOR TIMBERS

All structural members, assemblies or framework in a building, in combination with the floors, walls and other structural parts of the building shall be capable of sustaining, with due stability and stiffness the whole dead and imposed loadings as specified in appropriate codes without exceeding the limits of relevant stresses specified in IS 883.

The worst combination and location of loads shall be considered for designs. Wind and seismic forces shall not be considered to act simultaneously. The design requirements may be satisfied either by calculation using laws of mechanics or by prototype testing.

3.6 DESIGN ASPECTS OF TIMBER BEAMS

A timber beam may consist of a single member or may be built up from two or more members, called built up beams.

Timber beams are designed to resist,

1. Bending strength
2. Maximum horizontal shear
3. Stress at the bearings
4. Deflection

3.6.1 EFFECTIVE SPAN

The effective span of beams and other flexural members shall be taken as the distance from supports plus one-half of the required length of bearing at each end except that for continuous beams and joists the span may be measured from centre of bearing at those supports over which the beam is continuous.

$$L_{eff} = \frac{Wall/Bearing\ thickness}{2} + \text{Clear span} + \frac{Wall/Bearing\ thickness}{2}$$

3.6.2 BENDING STRENGTH

Usual formula for flexural strength shall be,

$$Actual\ Bending\ Stress\ (f_{ab}) = \frac{Bending\ Moment}{Section\ Modulus} = \frac{M}{Z} \leq Permissible\ Bending\ stress\ (f_b)$$

3.6.3 FORM FACTORS

a) Rectangular Section, $K_3 = 0.81\ \frac{D^2+89400}{D^2+55000}$

Form factor shall not be applied to beams having depth less than or equal to 300 mm

b) Box beams and I-Beams, $K_4 = 0.8 + 0.8y\left(\frac{D^2+89400}{D^2+55000} - 1\right)$

$$y = p_1^2\left(6 - 8p_1 + 3p_1^2\right)(l - q_1) + q_1$$

c) Solid circular cross sections, $K_5 = 1.18$

d) Square cross sections, $K_6 = 1.414$

3.6.4 WIDTH OF TIMBER BEAM

$$Width\ of\ the\ Beam > 50\ mm\ or\ \frac{Span}{50}\ \text{(Whichever is greater)}$$

3.6.5 DEPTH OF TIMBER BEAM

$$Depth\ of\ the\ Beam < 3 * Width\ of\ the\ beam$$

3.6.6 STIFFENING

All flexural members having a depth exceeding three times its width and or a span exceeding fifty times its width or both shall be laterally restrained from twisting or buckling and the distance between such restraints shall not exceed 50 times its width.

3.6.7 SHEAR

- The maximum horizontal shear, when the load on a beam moves from the support towards the centre of the span, and the load is at a distance of three to four times the depth of the beam from the support, shall be calculated from the following general formula:

$$H = \frac{VQ}{Ib}$$

- For rectangular beam, $H = \frac{3V}{2bD}$
- For notched beams, $H = \frac{3VD}{2bD_1^2}$
- Notched at upper face (Compression) where e > D: $H = \frac{3VD}{2bD_1}$
- Notched at upper face (Compression) where e < D: $H = \frac{3VD}{2b\left[D - \left(\frac{D_2}{D}\right)e\right]}$
- For concentrated loads, $V = \frac{10C\,(l-x)\left(\frac{x}{D}\right)^2}{9l\left[2+\left(\frac{x}{D}\right)^2\right]}$
- For uniformly distributed loads, $V = \frac{W}{2}\left(1 - \frac{2D}{l}\right)$

3.6.8 BEARING STRESS

LENGTH AND POSITION OF BEARING

- At any bearing on the side grain of timber, the permissible stress in compression perpendicular to the grain, f_{cn} is dependent on the length and position of the bearing
- The permissible stresses given in Table 1 of IS 883 for compression perpendicular to the grain are also the permissible stresses for any length at the ends of members and for bearing 150 mm or more in length at any other position;
- For bearings less than 150 mm in length and located 75 mm or more from the end of a member the permissible stress perpendicular to grain may be multiplied by the modification factor K_7

Modification factor K_7 for Bearing stresses							
Length of Bearing (mm)	**15**	**25**	**40**	**50**	**75**	**100**	**150 or More**
K_7	1.67	1.40	1.25	1.20	1.13	1.10	1.00

- No allowance need be made for the difference in intensity of the bearing stress due to bending of a beam
- The bearing area should be calculated as the net area after allowance for the amount of wane as permitted in IS 1331 : 1975
- For bearing stress under a washer or a small plate, the same coefficient recommended in Table 7 may be taken for a bearing with a length equal to the diameter of the washer or the width of the small plate
- When the direction of stress is at an angle to the direction of the grain in any structural member, then the permissible bearing stress in that member shall be calculated by the following formula:

$$f_{c\theta} = \frac{f_{cp} * f_{cn}}{f_{cp}\, sin^2\theta + f_{cn}\, cos^2\theta}$$

3.6.9 DEFLECTION

Permissible Deflection,

$\delta = \frac{L}{240}$ (Flexural members) $\delta = \frac{L}{150}$ (Cantilevers)

$\delta = \frac{L}{360}$ (Flexural members supporting brittle materials like gypsum ceiling, slate tiles and asbestos sheets)

Actual Deflection		Deflection
Cantilever Beam	Point Load (W)	$\frac{WL^3}{3EI}$
	UDL (w)	$\frac{wL^4}{8EI}$
Simply supported Beam	Point Load (W)	$\frac{WL^3}{48EI}$
	UDL (w)	$\frac{5wL^4}{384EI}$

3.6.10 DESIGN PROCEDURE FOR DESIGN OF TIMBER BEAMS

Step 1: Effective span of the beam: (Clause 7.5.2 – Pg 11 – IS 883)

For Cantilever beams, $l_{eff} = \frac{Wall\ thickness}{2}$ + **Clear span of the beam**

For Simply supported beams, $l_{eff} = \frac{Wall\ thickness}{2}$ + **Clear span** + $\frac{Wall\ thickness}{2}$

Step 2: Load Calculation:

Total Load (w) = Dead Load + Live load + Floor finish

Step 3: Bending moment:

		Shear Force	Bending Moment
Cantilever Beam	Point Load (W)	**W**	**WL**
	UDL (w)	**wL**	$\frac{wL^2}{2}$
Simply supported Beam	Point Load (W)	$\frac{W}{2}$	$\frac{WL}{4}$
	UDL (w)	$\frac{wL}{2}$	$\frac{wL^2}{8}$

Step 4: Calculation of Section Modulus:

$$Section\ Modulus\ (Z) = \frac{Bending\ Moment}{Bending\ Stress} = \frac{M}{\sigma}$$

Step 5: Calculation of Dimension of beam:

Section Modulus (Z)	**Rectangular Section**	**Square section**
	$Z = \frac{bd^2}{6}$	$Z = \frac{d^3}{6}$

Step 6: Check for Width of the beam: (Clause 7.5.5 – Pg 12 – IS 883)

$$\textit{Width of the Beam} > 50\ mm\ or\ \frac{Span}{50}$$

Step 7: Check for Depth of the beam: (Clause 7.5.6 – Pg 12 – IS 883)

$$\textit{Depth of the Beam} < 3 * \textit{Width of the beam}$$

Step 8: Check for Shear stress: (Clause 7.5.7 – Pg 12 – IS 883)

Actual Shear stress $H = \frac{3V}{2bD}$ and $V = \frac{W}{2}\left(1 - \frac{2D}{l}\right)$ **(For UDL)**

Permissible Shear stress $Table\ 3 - Pg\ 10$

Step 9: Check for Bearing stress: (Clause 7.5.7 – Pg 12 – IS 883)

$$\textit{Bearing stress} = \frac{\textit{Bearing Force}}{\textit{Cross sectional Area}} = \frac{V}{A}$$

Permissible Shear stress $Table\ 1 - Pg\ 6$ (Comp Stress Perpendicular to grain)

Step 9: Check for Deflection: (Clause 7.5.7 – Pg 12 – IS 883)

Actual Deflection		**Deflection**
Cantilever Beam	Point Load (W)	$\frac{WL^3}{3EI}$
	UDL (w)	$\frac{wL^4}{8EI}$
Simply supported Beam	Point Load (W)	$\frac{WL^3}{48EI}$
	UDL (w)	$\frac{5wL^4}{384EI}$

Permissible Deflection, $\delta = \frac{L}{240}$

1. A timber beam having a clear span of 6m carrying uniformly distributed load of 15kN/m including self-weight of the beam. Assume beam to be made of deodar wood and E = 0.95 x 10^4 N/mm^2.

Given:

Clear span (l)	=	6m	=	6000 mm
Uniformly distributed load (w)			=	15 kN/m (Including Self-weight)
Material			=	Deodar wood
Young's Modulus of wood (E)			=	0.95 x 10^4 N/mm^2

Solution:

Step 1: Effective span of the beam: (Clause 7.5.2 – Pg 11 – IS 883)

Consider the beam as simply supported beam with bearings of 250mm

For Simply supported beams, $L_{eff} = \frac{Wall\ thickness}{2} + \text{Clear span} + \frac{Wall\ thickness}{2}$

L_{eff} = (250/2)+6000+(250/2)

L_{eff} = 6250 mm (or) 6.25 m

Step 2: Load Calculation:

Total Load (w) = 15 kN/m (Includes Dead load and Live load)

Step 3: Bending moment and Shear Force:

		Shear Force (kN)	Bending Moment (kNm)
Simply supported Beam	UDL (w)	$\frac{wL}{2} = \frac{15*6.25}{2} = 46.875$	$\frac{wL^2}{8} = \frac{15*6.25^2}{8} = 73.24$

Step 4: Calculation of Section Modulus: (Clause 7.5.3 – Pg 11 – IS 883)

$$Section\ Modulus\ (Z) = \frac{Bending\ Moment}{Bending\ Stress} = \frac{M}{\sigma}$$

Bending Moment = 73.24 kNm

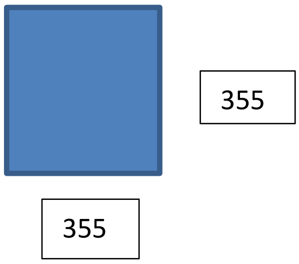
Bending Stress = 10.2 N/mm^2 (Table 1 – Pg 6)

$$\text{Section Modulus (Z)} = \frac{73.24 \text{ x } 1000 \text{ x } 1000}{10.2} = 7.18 \text{ x } 10^6 \text{ mm}^3$$

$$\text{Section Modulus (Z)} = 7.18 \text{ x } 10^6 \text{ mm}^3$$

Step 5: Calculation of Dimension of beam:

Consider Square cross section of the beam, $Z = \frac{d^3}{6}$



$$7.18 \text{ x } 10^6 = \frac{d^3}{6}$$

$$d^3 = 7.18 \text{ x } 10^6 \text{x } 6$$

d = 350.55 mm = 351 mm

Breadth = Depth = 355 mm

Size of the beam is 355 x 355 mm

Step 6: Check for Width of the beam: (Clause 7.5.5 – Pg 12 – IS 883)

$$\textit{Width of the Beam} > 50 \textit{ mm or } \frac{Span}{50}$$

As per IS 883, Width of the beam = 50 mm

Width of the beam = Span/50 = 6250/50 = 125 mm

Width of the beam = 125 mm

As per calculation, the width of the beam = 355 mm > 125 mm

Hence the chosen section is Okay.

Step 7: Check for Depth of the beam: (Clause 7.5.6 – Pg 12 – IS 883)

$$\textit{Depth of the Beam} < 3 * \textit{Width of the beam}$$

As per IS 883, Depth of the beam = 3*b = 3*355 = 1065 mm

As per calculation, Depth of the beam = 355 mm < 1065 mm

Hence the chosen section is Okay.

Step 8: Check for Shear stress: (Clause 7.5.7 – Pg 12 – IS 883)

Shear force $V = \frac{W}{2}\left(1 - \frac{2D}{l}\right)$

$V = \frac{(15*6.25)}{2}\left(1 - \frac{2*0.355}{6.25}\right)$ V = 41.55 kN

Actual Shear stress $H = \frac{3V}{2bD}$

$H = \frac{3*41.55*1000}{2*355*355}$

Actual Shear stress, H = 0.4945 N/mm^2

Permissible Shear stress, = 0.7 N/mm^2 *Table 1 – Pg 6*

Actual Shear stress < Permissible shear stress

Hence the section is Okay

Step 9: Check for Bearing stress: (Clause 7.5.8 – Pg 13 – IS 883)

$$Bearing\ stress = \frac{Bearing\ Force}{Cross\ sectional\ Area} = \frac{V}{A}$$

Bearing stress $= \frac{46.875*1000}{355*355} =$ 0.371 N/mm^2

Permissible Bearing stress = 2.7 N/mm^2 *Table 1 – Pg 6*

(Comp Stress Perpendicular to grain)

Actual Bearing stress < Permissible bearing stress

Hence the section is Okay

Step 9: Check for Deflection: (Clause 7.5.9 – Pg 13 – IS 883)

Actual deflection, $\delta = \frac{5wL^4}{384EI} = \frac{5*15*6250^4}{384*0.95*10^4*\left(\frac{355*355^3}{12}\right)} =$ 23.70 mm

Permissible Deflection, $\delta = \frac{L}{240} = \frac{6250}{240} =$ 26.05 mm

Actual Deflection < Permissible Deflection

Hence the section is Okay

2. Floor beams of group B timber are spaced at 1.2m centre on a clear span of 3m with a wall thickness of 20cm at each end. The dead load of roof and beam is 2500 N/m^2 and live load is 1500 N/m^2. Design the beam for inside location. E = 11000 N/mm^2.

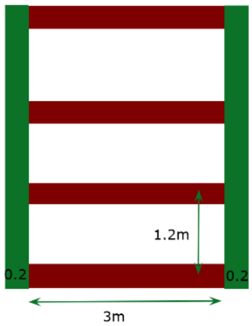
Given:

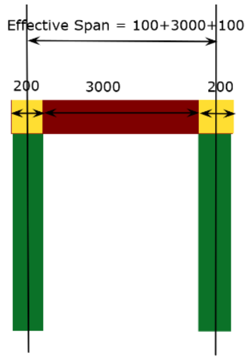
Clear span (l)	=	3m	=	3000 mm
Spacing of beams			=	1.2 m c/c
Thickness of wall	=	20 cm	=	200mm
Dead load			=	2.5 kN/m^2
Live load			=	1.5 kN/m^2
Young's Modulus of wood		(E)	=	11000 N/mm^2
Group of Wood			=	B
Location of the beam			=	Inside Location

Solution:

Step 1: Effective span of the beam: (Clause 7.5.2 – Pg 11 – IS 883)

$$L_{eff} = \frac{t_w}{2} + \text{Clear span} + \frac{t_w}{2}$$





L_{eff} = (200/2)+3000+(200/2)

L_{eff} = 3200 mm (or) 3.2 m

Step 2: Load Calculation:

Dead Load	= 2.5 * 1.2	= 3 kN/m
Live Load	= 1.5 * 1.2	= 1.8 kN/m
Total Load	(w)	= 4.8 kN/m

Step 3: Bending moment and Shear Force:

		Shear Force (kN)	Bending Moment (kNm)
Simply supported Beam	UDL (w)	$\frac{wL}{2} = \frac{4.8 * 3.2}{2}$ = 7.68	$\frac{wL^2}{8} = \frac{4.8 * 3.2^2}{8}$ = 6.14

Step 4: Calculation of Section Modulus: (Clause 7.5.3 – Pg 11 – IS 883)

$$\textbf{Section Modulus } (Z) = \frac{\textbf{Bending Moment}}{\textbf{Bending Stress}} = \frac{M}{\sigma}$$

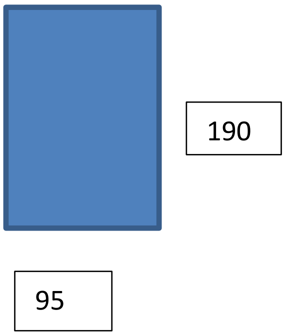
Bending Moment = 6.14 kNm

Bending Stress = 12 N/mm^2 (Table 3 – Pg 10)

$$\text{Section Modulus (Z)} = \frac{6.14 * 1000000}{12} = 0.511 \times 10^6 \text{ mm}^3$$

Step 5: Calculation of Dimension of beam:

Consider Rectangular cross section of the beam, Assume d = 2b



$$Z = \frac{bd^2}{6}$$

$$0.511 \times 10^6 = \frac{b*(2b)^2}{6}$$

$$0.511 \times 10^6 = \frac{4b^3}{6}$$

$$b^3 = \frac{0.511 \times 10^6 \times 6}{4}$$

b = 91.52 mm = 95 mm

Breadth (b) = 95 mm

Depth (d) = 2b = 2*95 = 190 mm

Size of the beam is 95 x 190 mm

Step 6: Check for Width of the beam: (Clause 7.5.5 – Pg 12 – IS 883)

$$\textbf{Width of the Beam} > \textbf{50 mm or } \frac{\textbf{Span}}{\textbf{50}}$$

As per IS 883, Width of the beam = 50 mm

Width of the beam = Span/50 = 3200/50 = 64 mm

As per calculation, the width of the beam = 95 mm > 64 mm

Hence the chosen section is Okay.

Step 7: Check for Depth of the beam: (Clause 7.5.6 – Pg 12 – IS 883)

$$\textbf{Depth of the Beam} < 3 * \textbf{Width of the beam}$$

As per IS 883, Depth of the beam = 3*b = 3*95 = 275mm

As per calculation, Depth of the beam = 190 mm < 275 mm

Hence the chosen section is Okay.

Step 8: Check for Shear stress: (Clause 7.5.7 – Pg 12 – IS 883)

Shear force $V = \frac{W}{2}\left(1 - \frac{2D}{l}\right)$ **W = wL**

$$V = \frac{(4.8 * 3.2)}{2}\left(1 - \frac{2 * 0.19}{3.2}\right)$$

V = 6.76 kN

Actual Shear stress $H = \frac{3V}{2bD}$

$$H = \frac{3*6.76*1000}{2*95*190}$$

Actual Shear stress, H = 0.567 N/mm^2

Permissible Shear stress, = 0.64 N/mm^2 ***Table 3 – Pg 10***

Actual Shear stress < Permissible shear stress

Hence the section is Okay

Step 9: Check for Bearing stress: (Clause 7.5.8 – Pg 13 – IS 883)

$$\text{Bearing stress} = \frac{\text{Bearing Force}}{\text{Cross sectional Area}} = \frac{V}{A}$$

Bearing stress $= \frac{7.68*1000}{95*190} = 0.42$ N/mm^2

Permissible Bearing stress = 2.5N/mm^2 ***Table 3 – Pg 10***

(Comp Stress Perpendicular to grain)

Actual Bearing stress < Permissible bearing stress

Hence the section is Okay

Step 9: Check for Deflection: (Clause 7.5.9 – Pg 13 – IS 883)

w = 4.8 kN/m = (4.8 * 1000 N)/ (1000 mm) = 4.8 N/mm

Actual deflection, $\delta = \frac{5wL^4}{384EI} = \frac{5*4.8*3200^4}{384*11000*\left(\frac{95*190^3}{12}\right)}$ = **10.97** mm

Permissible Deflection, $\delta = \frac{L}{240} = \frac{3200}{240} =$ 13.33 mm

Actual Deflection < Permissible Deflection

Hence the section is Okay

UNIT 4

DESIGN OF TIMBER COLUMNS

4.1 INTRODUCTION

Timber is one of the most useful and important material for constructions. Selecting timber is not an easy task, because timber has different types out of which selecting the right material is an important key. Timber is an expensive material to be incorporated in a building for different purpose therefore it should necessarily be strong, tough and durable. Timber doors or windows and etc. contribute a lot in the beautification and overall look of interiors. Timber is used in doors, windows, cabinet, cupboards, shelves, tables and railings etc. Timber is also popularly used in the form of plywood & raw wood. Products like ply blocks and ply boards. Heavy patterned doors and windows are made of solid wood/Timber to provide the strength, toughness and durability. Type of timber to be used for right purpose is important because if timber used in construction is of low quality then this may need replacement. While selecting timber one should consider its quality aspect as timber must be free from decay like rotten, fungi and termite.

4.1 DESIGN ASPECTS OF TIMBER COLUMNS

4.1.1 SOLID COLUMNS

4.1.1.1 CLASSIFICATION OF SOLID COLUMNS

Solid column shall be classified into short, intermediate and long columns depending upon their slenderness ratio (S/d) as follows:

Short Column $\frac{S}{d} < 11$

Intermediate Column $11 < \frac{S}{d} < K_8$ $\qquad K_8 = 0.584\sqrt{\frac{E}{f_{cp}}}$

Long Column $\frac{S}{d} > K_8$

4.1.1.2 PERMISSIBLE COMPRESSIVE STRESS IN SOLID COLUMNS

Short Column $f_c = f_{cp}$

Intermediate Column $f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{s}{K_8\, d}\right)^4\right]$

Long Column $f_c = \frac{0.329\, E}{\left(\frac{S}{d}\right)^2}$

- In case of solid columns of timber S/d shall not exceed 50
- The formulae given are for columns with pin end conditions and length shall be suitably modified with other end conditions
- The permissible load on a column of circular cross-section shall not exceed that permitted for a square column of an equivalent cross-sectional area
- For determining S/d ratio of a tapered column, its least dimension. shall be taken as the sum of the corresponding least dimensions at the small end of the column and one-third of the difference between this least dimension at the small end and the corresponding least dimension at the large end, but in no case shall the least dimension for the column be taken as more than one and a half times the least dimension at the small end. The induced stress at the small end of the tapered column shall not exceed the permissible compressive stress in the direction of grain.

4.1.2 BOX AND BUILT-UP COLUMNS

4.1.2.1 CLASSIFICATION OF BOX AND BUILT-UP COLUMNS

Solid column shall be classified into short, intermediate and long columns depending upon their slenderness ratio (S/d) as follows:

Short Column $$\frac{S}{\sqrt{d_1^2 + d_2^2}} < 8$$

Intermediate Column $$8 < \frac{S}{\sqrt{d_1^2 + d_2^2}} < K_9 \qquad K_9 = \frac{\pi}{2}\sqrt{\frac{U\,E}{5\,q\,f_{cp}}}$$

Long Column $$\frac{S}{\sqrt{d_1^2 + d_2^2}} > K_9$$

4.1.2.2 PERMISSIBLE COMPRESSIVE STRESS IN BOX AND BUILT-UP COLUMNS

Short Column $$f_c = q\,f_{cp}$$

Intermediate Column $$f_c = q\,f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_9\sqrt{d_1^2 + d_2^2}}\right)^4\right]$$

Long Column $$f_c = \frac{0.329\,E}{\left(\frac{S}{\sqrt{d_1^2 + d_2^2}}\right)^2}$$

4.1.2.3 CONSTANTS U AND q

The following are the values of U and q depending on the plank thickness (t)

t (mm)	U	q
25	0.80	1.00
50	0.60	1.00

4.1.3 SPACED COLUMNS

The formulae for solid columns as specified before are applicable to spaced columns with a restraint factor of 2.5 or 3 depending upon distance of end connectors in the column

A restrained factor of 2.5 for location of centroid group of fasteners at S/20 from end and 3 for location at S/10 to S/20 from end shall be taken

4.1.3.1 PERMISSIBLE COMPRESSIVE STRESS IN SPACED COLUMNS

Intermediate Column

$$f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_{10}\, d}\right)^4\right]$$

$$K_{10} = 0.584\sqrt{\frac{2.5\,E}{f_{cp}}}$$

Long Column

$$f_c = \frac{0.329\,E * 2.5}{\left(\frac{S}{d}\right)^2}$$

- For individual member of spaced column S/d ratio shall not exceed 80
- Compression members shall not he notched. When it is necessary to pass services through such a ember, this shall be effected by mean2 of a bored hole provided that the local stress is calculated and found to be within the permissible stress specified. The distance from the edge of the hole to the edge of the member shall not be less than one-quarter of width of the face.

4.1.4 STRUCTURAL MEMBERS SUBJECT TO BENDING AND AXIAL STRESSES

- Structural members subjected both to bending and axial compression shall be designed to comply with the following formula:

$$\frac{f_{ac}}{f_c} + \frac{f_{ab}}{f_b} \ngtr 1$$

- Structural members subjected both to bending and axial tension shall be designed to comply with the following formula:

$$\frac{f_{at}}{f_t} + \frac{f_{ab}}{f_b} \ngtr 1$$

4.1.5 DESIGN PROCEDURE FOR DESIGN OF TIMBER COLUMN

4.1.5.1 DETERMINATION OF SAFE LOAD

Step 1: Slenderness Ratio: (Clause 7.6.1 – Pg 13 & 14 – IS 883)

$\lambda = \frac{S}{d} < 11$ For Short Column,

$\lambda = \frac{S}{d} < 11 \ to \ K_8$ For Intermediate Column,

$\lambda = \frac{S}{d} > K_8$ For Long Column,

$$K_8 = 0.584\sqrt{\frac{E}{f_{cp}}}$$

Step 2: Calculation of Safe stress : (Clause 7.6.1 – Pg 14 – IS 883)

Calculate the Permissible or safe stress (f_c)

For Short Columns, Calculate $f_c = f_{cp}$ from Table 1 or Table 3 of IS 883

For intermediate columns, $f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_8 d}\right)^4\right]$

For long column, $f_c = \frac{0.329\,E}{\left(\frac{S}{d}\right)^2}$

Step 3: Calculation of Safe Axial Load:

$$Safe\ Axial\ Load\ (P) = \frac{Safe\ stress}{Cross\ sectional\ area} = \frac{f_c}{A}$$

4.1.5.2 DESIGN OF SOLID COLUMNS

Step 1: Dimension of the column :

Assume it as short/Intermediate/Long column

If it is assumed as short column $\frac{S}{d} = 11$

Step 2: Calculation of Safe stress : (Clause 7.6.1 – Pg 14 – IS 883)

Calculate the Permissible or safe stress (f_c)

For Short Columns, Calculate $f_c = f_{cp}$ from Table 1 or Table 3 of IS 883

For intermediate columns, $f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_8 d}\right)^4\right]$

For long column, $f_c = \frac{0.329\,E}{\left(\frac{S}{d}\right)^2}$

Step 3: Calculation of Safe stress : (Clause 7.6.1 – Pg 14 – IS 883)

Permissible safe stress (f_c) From step 2

Actual safe stress = $\frac{Load}{Cross\ sectional\ Area}$

Step 4: Calculation of Slenderness ratio:

$\lambda = \frac{S}{d} < 11$ For Short Column,

$\lambda = \frac{S}{d} < 11\ to\ K_8$ For Intermediate Column,

$\lambda = \frac{S}{d} > K_8$ For Long Column,

$$K_8 = 0.584\sqrt{\frac{E}{f_{cp}}}$$

4.1.5.3 DESIGN OF BOX AND BUILT-UP COLUMNS

Step 1: Dimension of the column :

Assume it as short/Intermediate/Long column

If it is assumed as short column $\frac{S}{\sqrt{d_1^2 + d_2^2}} = 8$

Step 2: Calculation of Safe stress : (Clause 7.6.1 – Pg 14 – IS 883)

Calculate the Permissible or safe stress (f_c)

For Short Columns, Calculate $f_c = q\, f_{cp}$ from Table 1 or Table 3 of IS 883

For intermediate columns, $f_c = q\, f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_9\sqrt{d_1^2 + d_2^2}}\right)^4\right]$

For long column, $f_c = \frac{0.329\,E}{\left(\frac{S}{\sqrt{d_1^2 + d_2^2}}\right)^2}$

Step 3: Calculation of Safe stress : (Clause 7.6.1 – Pg 14 – IS 883)

Permissible safe stress (f_c) From step 2

Actual safe stress = $\frac{Load}{Cross\ sectional\ Area}$

Step 4: Calculation of Slenderness ratio:

Short Column $\frac{S}{\sqrt{d_1^2 + d_2^2}} < 8$

Intermediate Column $8 < \frac{S}{\sqrt{d_1^2 + d_2^2}} < K_9$

Long Column $\frac{S}{\sqrt{d_1^2 + d_2^2}} > K_9$

$$K_9 = \frac{\pi}{2}\sqrt{\frac{U\,E}{5\,q\,f_{cp}}}$$

3. A timber column 200 mm x 200 mm having an unsupported length of 2m. Assuming the column to be of Sal wood. Find the safe axial load.

Given:

Breadth of the column (b)	=	200 mm		
Depth of the column (d)	=	200 mm		
Unsupported length (S)	=	2m	=	2000 mm
Material of Timber	=	Sal wood		

Solution:

Step 1 : Slenderness ratio : (Clause 7.6.1 – Pg 13 & 14 – IS 883)

$$\lambda = \frac{S}{d} = \frac{2000}{200} = 10$$

$$\frac{S}{d} = 10 < 11$$

Hence the column can be classified as Short column

Step 2 : Calculation of Safe Stress : (Clause 7.6.1.1 – Pg 14 – IS 883)

For Short Columns, Calculate $f_c = f_{cp} = 10.6$ N/mm^2 from Table 1 of IS 883

Step 3 : Calculation of Safe Load :

$$Safe\ stress\ (f_c) = \frac{Safe\ Load}{Cross\ sectional\ area} = \frac{P}{A}$$

$$10.6 = \frac{P}{200 * 200}$$

P = 10.6*200*200

P = 42.4 x 10^4 N

Safe Load (P) = 424 kN

4. A timber column is made of Deodar wood and is 200 mm diameter. The effective length of the column is 1.25m. Find the safe axial load.

Given:

Diameter of the column (D)= 200 mm

Effective length (S) = 1.25m = 1250 mm

Material of Timber = Deodar wood

Solution:

Step 1 : Equivalent dimension of Circular column as Square (Clause 7.6.1.6 – Pg 14 – IS 883)

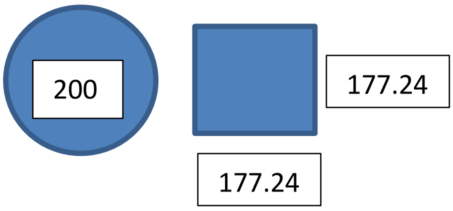
In case of circular column, the area of circular column should be converted to equivalent area of a square column.

Area of circle = Area of square

$$\frac{\pi}{4} * D^2 = B * B$$

$$\frac{\pi}{4} * 200^2 = B^2$$

$$B = 177.24 \text{ mm}$$



Step 2 : Slenderness ratio : (Clause 7.6.1 – Pg 13 & 14 – IS 883)

$$\lambda = \frac{S}{d} = \frac{1250}{177.24} = 7.05 \qquad \frac{S}{d} = 7.05 < 11$$

Hence the column can be classified as Short column

Step 3 : Calculation of Safe Stress : (Clause 7.6.1.1 – Pg 14 – IS 883)

For Short Columns, $f_c = f_{cp} = 7.8 \text{ N/mm}^2$ from Table 1 of IS 883

Step 4 : Calculation of Safe Load :

$$Safe\ stress\ (f_c) = \frac{Safe\ Load}{Cross\ sectional\ area} = \frac{P}{A}$$

$$7.8 = \frac{P}{\frac{\pi}{4} * 200^2} \quad \text{or} \quad 7.8 = \frac{P}{177.24 * 177.24}$$

Safe Load (P) = 245.04 kN

5.A timber column is 200 mm x 300 mm in section having an unsupported length of 3.5m. Find the safe axial load for the column assuming it to be of Sal wood. Take E = 1.08 x 10^4 N/mm^2.

Given:

Breadth of the column (b)	=	200 mm
Depth of the column (d)	=	300 mm
Unsupported length (S)	=	3.5m = 3500 mm
Material of Timber	=	Sal wood
Young's Modulus of wood (E)	=	1.08 x 10^4 N/mm^2

Solution:

Step 1 : Slenderness ratio : (Clause 7.6.1 – Pg 13 & 14 – IS 883)

$$\lambda = \frac{S}{d} = \frac{3500}{200} = 17.5 \quad > 11$$

Hence the column may be intermediate or long column

f_{cp} = 10.6 N/mm^2 from Table 1 of IS 883

$$K_8 = 0.584\sqrt{\frac{E}{f_{cp}}} = 0.584\sqrt{\frac{10800}{10.6}} = 18.64$$

Since S/d = 17.5 lies between 11 and k8 = 18.64, the column is classified as Intermediate column

Step 2 : Calculation of Safe Stress : (Clause 7.6.1.2 – Pg 14 – IS 883)

For Intermediate Columns, $f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_8 d}\right)^4\right] = 10.6\left[1 - \frac{1}{3}\left(\frac{3500}{18.64*200}\right)^4\right] = 7.85$ N/mm^2

Step 3 : Calculation of Safe Load :

$$Safe\ stress\ (f_c) = \frac{Safe\ Load}{Cross\ sectional\ area} = \frac{P}{A}$$

$$7.85 = \frac{P}{200 * 300}$$

Safe Load (P) = 471 kN

6. A timber column 200 mm x 400 mm in section is having an unsupported length of 4.75m. Find the safe load assuming the column to be of Sal wood. Take E = 1.08 x 10^4 N/mm^2.

Given:

Breadth of the column (b)	=	200 mm		
Depth of the column (d)	=	400 mm		
Unsupported length (S)	=	4.75m	=	4750 mm
Material of Timber	=	Sal wood		
Young's Modulus of wood (E)	=	1.08 x 10^4 N/mm^2		

Solution:

Step 1 : Slenderness ratio : (Clause 7.6.1 – Pg 13 & 14 – IS 883)

$$\lambda = \frac{S}{d} = \frac{4750}{200} = 23.75 \quad > \quad 11$$

Hence the column may be intermediate or long column

f_{cp} = 10.6 N/mm^2 from Table 1 of IS 883

$$K_8 = 0.584\sqrt{\frac{E}{f_{cp}}} = 0.584\sqrt{\frac{10800}{10.6}} = 18.64$$

$\frac{S}{d} = 23.75 > K8 = 18.64$ The column is classified as Long column

Step 2 : Calculation of Safe Stress : (Clause 7.6.1.1 – Pg 14 – IS 883)

For Long Columns, $f_c = \frac{0.329\,E}{\left(\frac{S}{d}\right)^2} = \frac{0.329*10800}{(23.75)^2} = 6.299$ N/mm^2

Step 3 : Calculation of Safe Load :

$$Safe\ stress\ (f_c) = \frac{Safe\ Load}{Cross\ sectional\ area} = \frac{P}{A}$$

$$6.299 = \frac{P}{200 * 400}$$

Safe Load (P) = 503.92 kN

7. Design a solid square column to carry an axial load of 450 kN. The column material is Deodar wood which is provided for outside location. The effective length of the column is 3m.

Given:

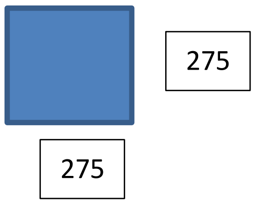
Axial load on the column (P)	=	450 kN		
Unsupported length (S)	=	3m	=	3000 mm
Material of Timber	=	Deodar wood		
Location of the member	=	Outside location		

Solution:

Step 1 : Dimensioning of column : (Clause 7.6.1 – Pg 13 & 14 – IS 883)

Let us assume the column to be a short column

$\frac{S}{d} < 11$ $\qquad \frac{3000}{d} = 11$ $\qquad d = 272.72$ mm



Provide d = 275 mm

Size of the column is 275 x 275 mm

Step 2 : Calculation of Permissible Safe Stress : (Clause 7.6.1.1 – Pg 14 – IS 883)

For Short Columns, $f_c = f_{cp} = 6.9$ N/mm^2 from Table 1 of IS 883

Step 3 : Check for Actual Safe stress:

$$Safe\ stress\ (f_c) = \frac{Load}{Cross\ sectional\ area} = \frac{P}{A} \qquad = \frac{450*1000}{275*275}$$

$$f_c = 5.95 \text{ N/mm}^2$$

Permissible safe stress > Actual safe stress

Hence the section is safe

Step 4 : Check for Slenderness ratio : (Clause 7.6.1 – Pg 13 & 14 – IS 883)

$$\lambda = \frac{S}{d} = \frac{3000}{275} = 10.91 < 11$$

Then column is short column and safe

8. Design a rectangular column of group B to be used in a open shed to carry an axial load of 550 kN. The effective length of the column is 3m.

Given:

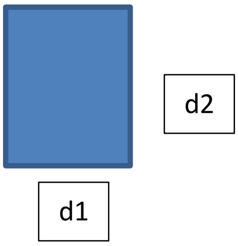
Axial load on the column (P)	=	550 kN		
Effective length (S)	=	3m	=	3000 mm
Group of Timber	=	B		
Location of timber	=	Open shed		

Solution:

Step 1 : Dimensioning of column (d1): (Clause 7.6.1 – Pg 13 & 14 – IS 883)

Let us assume the column to be a short column

$$\frac{S}{d1} < 11$$



$$\frac{3000}{d} = 11$$

$$d1 = 272.72 \text{ mm}$$

Provide d1 = 275 mm

Step 2 : Dimensioning of column (d2):

From Table 3 of IS 883, f_{cp} = 7.8 N/mm^2 (Inside location)

For open shed (Outside location) $f_{cp} = \frac{5}{6} * 7.8 = 6.5$ N/mm^2

For Short Columns, $f_c = f_{cp} = 6.5$ N/mm^2

$$Safe\ stress\ (f_c) = \frac{Load}{Cross\ sectional\ area}$$

$$6.5 = \frac{550 * 1000}{d1 * d2}$$

$$6.5 = \frac{550 * 1000}{275 * d2}$$

$$d_2 = \frac{550 * 1000}{275 * 6.5}$$

d_2 = 307.69 mm

Provide d_2 = 310 mm

Size of the rectangular column is 275 x 310 mm

Step 3 : Check for Actual Safe stress:

For Short Columns, Permissible stress $f_c = f_{cp}$ = 6.5 N/mm^2 from step (2)

$$\boldsymbol{Actual\ stress\ (f_c) = \frac{Load}{Cross\ sectional\ area}}$$

$$f_c = \frac{550 * 1000}{275 * 310}$$

f_c = 6.45 N/mm^2

Permissible safe stress > Actual safe stress

Hence the section is safe

Step 4 : Check for Slenderness ratio : (Clause 7.6.1 – Pg 13 & 14 – IS 883)

$$\boldsymbol{\lambda = \frac{S}{d}} = \frac{3000}{275} = 10.91 < 11$$

Then column is short column and safe

9. Design a 5m long rectangular box columns built up by 50mm thick deodar planks to carry an axial load of 400 kN

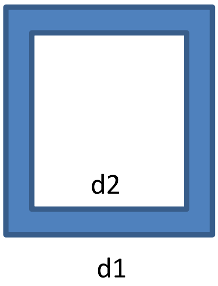
Given:

Axial load on the column (P)	=	400 kN		
Length of the column (S)	=	5m	=	5000 mm
Thickness of Planks	=	50 mm		
Material of timber	=	Deodar		

Solution:

Step 1 : Dimensioning of column (Shorter length): (Clause 7.6.2.1 – Pg 13 & 14 – IS 883)

Let us assume the column to be a short column



$$\frac{S}{\sqrt{d_1^2 + d_2^2}} < 8$$

Width of the box column, $d_1 = d_2$ +thickness of plank on both sides of the core

$$d_1 = d_2 + 50 + 50 = d_2 + 100$$

$$\frac{5000}{\sqrt{d_1^2 + d_2^2}} = 8$$

$$\sqrt{d_1^2 + d_2^2} = \frac{5000}{8} = 625$$

Squaring on both sides, $d_1^2 + d_2^2 = 390625$

$$(d_2 + 100)^2 + d_2^2 = 390625$$

$$d_2^2 + 200d_2 + 10000 + d_2^2 = 390625$$

$$2d_2^2 + 200d_2 - 380625 = 0$$

$$d_2 = 389.10 \text{ mm}$$

Provide $d_2 = 400$ mm

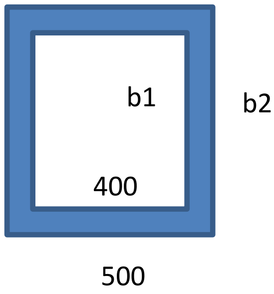
$d_1 = d_2 + 100 = 400 + 100$ $d_1 = 500$ mm

Step 2 : Dimensioning of column (Longer length):

From Table 1 of IS 883, f_{cp} = 7.8 N/mm^2 (Inside location)

For 50mm thick plank, q = 1 (Cl 7.6.1.2 – Pg 14 of IS 883)

For Short Columns, $f_c = q\ f_{cp}$ = 7.8 N/mm^2



$$Safe\ stress\ (f_c) = \frac{Load}{Cross\ sectional\ area}$$

$$7.8 = \frac{400 * 1000}{d1 * b1}$$

$$7.8 = \frac{400 * 1000}{400 * b1}$$

$$b_1 = \frac{400 * 1000}{400 * 7.8}$$

b_1 = 128 mm

Provide b_1 = 150 mm

$b_2 = b_1 + 100 = 150 + 100$

b_2 = 250 mm

Size of the rectangular box column is 500 x 250 mm

Step 3 : Check for Actual Safe stress:

For Short Columns, Permissible stress $f_c = f_{cp}$ = 7.8 N/mm^2 from step (2)

$$Actual\ stress\ (f_c) = \frac{Load}{Cross\ sectional\ area} = \frac{400*1000}{250*500}$$

f_c = 3.2 N/mm^2

Permissible safe stress > Actual safe stress. Hence safe

Step 4 : Check for Slenderness ratio : (Clause 7.6.2.1 – Pg 13 & 14 – IS 883)

$$\lambda = \frac{S}{\sqrt{d_1^2 + d_2^2}} = \frac{5000}{\sqrt{500^2+400^2}} = 7.80 < 8$$

Then column is short column and safe

10. A spaced compression member is 3m long between the end connections and carries an axial load of 65000 N. The wood used is Dhaman for which f_c = 9.1 N/mm^2 and E = 12000 N/mm^2. Design the member with end connections using 50 mm thick planks.

Given:

Axial load on the column (P)	=	65000 N		
Length of the column (S)	=	3m	=	3000 mm
Thickness of Planks	=	50 mm		
Material of timber	=	Dhaman		
Young's Modulus of wood (E)	=	12000 N/mm^2		
Compressive stress (f_c)	=	9.1 N/mm^2		

Solution:

Step 1 : Classification of Column :

Assuming the end distance less than l/20,

Slenderness ratio, $\lambda = \frac{S}{d} = \frac{3000}{50} = 60$

From Table 1 of IS 883, f_c = 9.1 N/mm^2 (Inside location)

$$K_8 = 0.584\sqrt{\frac{E}{f_{cp}}} \qquad = 0.584\sqrt{\frac{12000}{9.1}} = 21.207$$

$\frac{S}{d} = 60 > \mathrm{K8} = 21.20$ The co400*400lumn is classified as Long column

Step 2 : Calculation of Permissible Safe Stress : (Clause 7.6.3.2 – Pg 14 – IS 883)

For Long Columns, $f_c = \frac{0.329\,E*2.5}{\left(\frac{S}{d}\right)^2} = \frac{0.329*12000*2.5}{(60)^2}$ = 2.74 N/mm^2

Step 3 : Area of timber required:

$$Safe\ stress\ (f_c) = \frac{Load}{Cross\ sectional\ area} = \frac{P}{A} \qquad 2.74 = \frac{65000}{A}$$

Area Required = 23722.63 mm^2

Assume, Provide 3 Planks with spacer and end blocks of 50mm thick

Area Required = 23722.63 mm^2

3 x (B x t) = 23722.63

3(B x 50) = 23722.63

B = 158 mm

Hence Size of the plank is 3# of 50 * 160 mm

Spacer block are provided at center of the member of 50 x 100 x 100 mm

Permissible safe stress > Actual safe stress

Hence the section is safe

UNIT 5

ANALYSIS AND DESIGN OF TIMBER TRUSSES

5.1 INTRODUCTION

Basically, a structure is defined as an assembly of bars or beams fastened together by bolted, riveted or welded joints, designed to support the external forces. The members of any structure are fastened together in such a way that any movement between them on application of load is prevented. A structure is basically divided into (a) Frames and (b) Trusses. Analysis of plane trusses is dealt in this chapter.

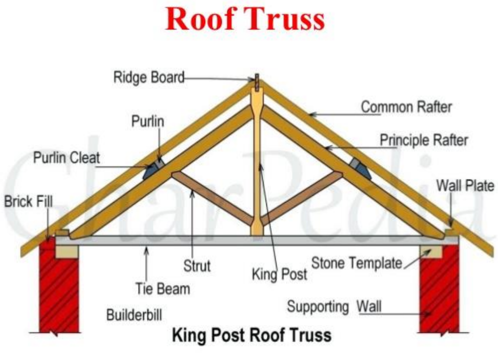
5.2 FRAME AND TRUSS

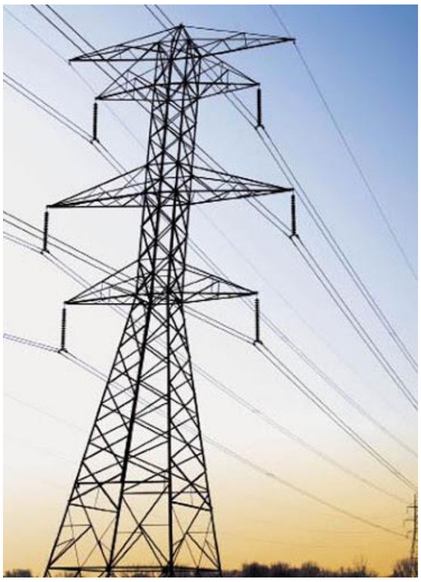
Frames are the fully constrained structures which are designed to support loads and are stationary structures. It consists of several bars or members with one or more than one of its members is subjected to more than two forces.

Trusses are also fully constrained structures, which are designed to support the stationary or moving loads. It is a system of uniform bars or members, riveted or welded at their ends and the loads ae applied only at the joints.

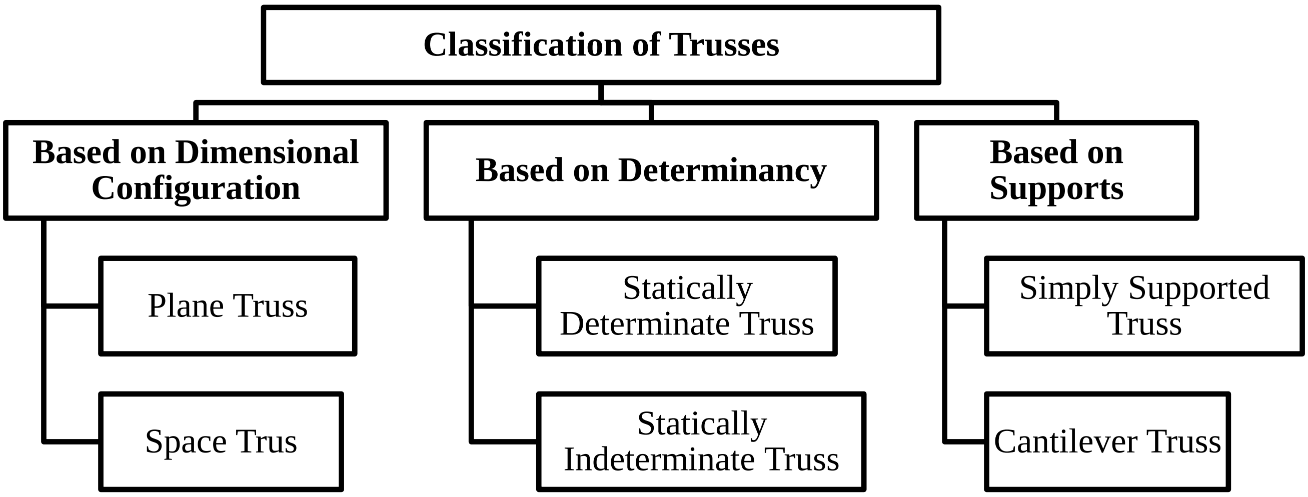
5.2.1 OTHER FORMS OF TRUSSES

- Truss : A pin jointed frame used to support roof
- Girder : Beam in bridge
- Tower : Used as Masts



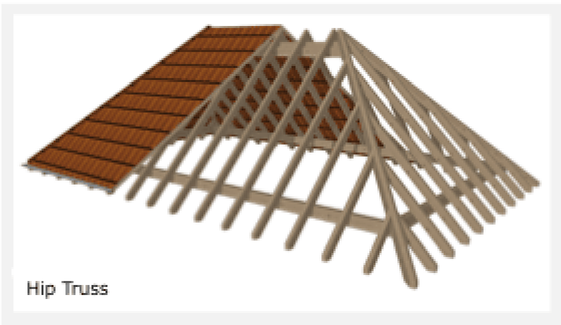


5.3 CLASSIFICATION OF TRUSSES



PLANE TRUSS A truss of which the members lie in a single plane (i.e., two-dimensional configuration) is called plane truss. A basic plane truss is of three members, forming a triangle

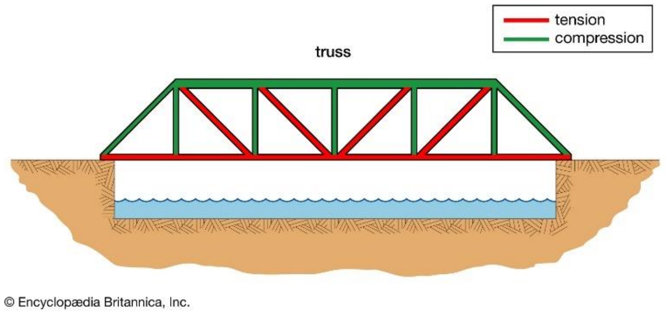
SPACE TRUSS A truss of which the members lie in different plane (i.e., three-dimensional configuration) is called plane truss. A basic space truss consists of six bars joined at their ends to form a tetrahedron.

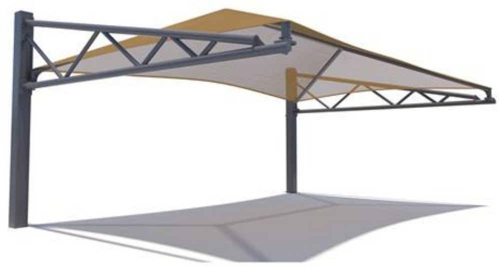


STATICALLY DETERMINATE TRUSS A truss which could be analyzed completely using the three static equilibrium equations alone (i.e., ΣH = 0, ΣV = 0, ΣM = 0) is called statically determinate truss.

STATICALLY INDETERMINATE TRUSS A truss which could not be analyzed by the static equilibrium equations alone, needs additional equations for the complete analysis is called statically indeterminate truss.

SIMPLY SUPPORTED TRUSS	The truss supported on a roller support at one end and hinged support on the other end is called simply supported truss. Ina simply supported truss, reactions at supports are necessary to analyze
CANTILEVER TRUSS	The truss which is fixed at one end on a wall or column and free at the other end is called a Cantilever truss. In a cantilever truss, the reactions at supports are not necessary to analyze the member forces





5.3.1 PERFECT FRAME

A frame which has just sufficient number of members to keep it in equilibrium without any change in its shape under the action of external loads is called a perfect frame (or rigid frame)

SIMPLY SUPPORTED TRUSS	CANTILEVER TRUSS
$m = 2j - 3$	$m = 2j - 4$
m = number of members	m = number of members
j = number of joints	j = number of joints in the truss including points of supports

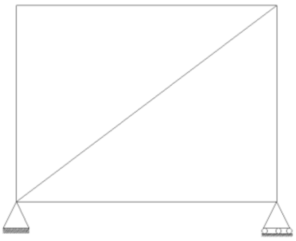
5.3.2 IMPERFECT FRAME

A frame which does not satisfy the expressions of perfect frame is called an imperfect frame. The imperfect frame is further classified into

- Deficient frame
- Redundant frame

DEFICIENT FRAME	Number of members in the frame is less than that of the members required to keep the frame in equilibrium $m < 2j - r$
REDUNDANT FRAME	Number of members is greater than that required for equilibrium $m > 2j - r$

1. Determine the state of Determinacy for the following:

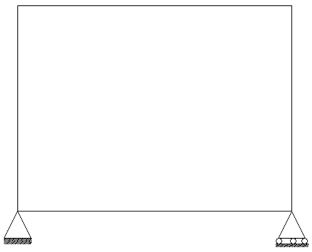


m = 5 $\quad$ **m = 2j - r**

j = 4 $\quad$ m = 2*4 – 3 = 5

r = 3 $\quad$ 5 = 5

It is a Perfect Truss

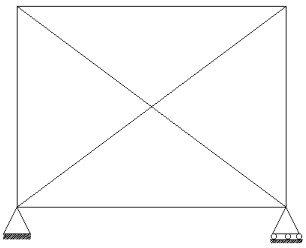


m = 4 $\quad$ **m = 2j - r**

j = 4 $\quad$ m = 2*4 – 3 = 5

r = 3 $\quad$ 4 < 5

It is a Deficient Truss

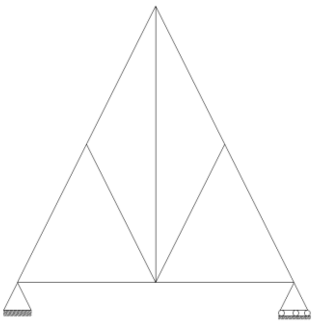


m = 6 $\quad$ **m = 2j - 3**

j = 4 $\quad$ m = 2*4 – 3 = 5

r = 3 $\quad$ 6 > 5

It is a Redundant Truss

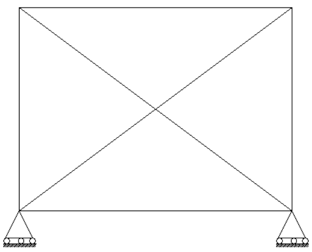


m = 9 $\quad$ **m = 2j - 3**

j = 6 $\quad$ m = 2*6 – 3 = 9

r = 3 $\quad$ 9 = 9

It is a Perfect Truss



m = 6 $\quad$ **m = 2j - 3**

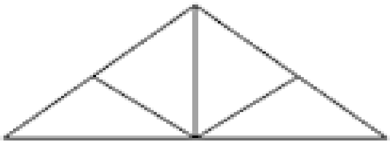
j = 4 $\quad$ m = 2*4 – 2 = 6

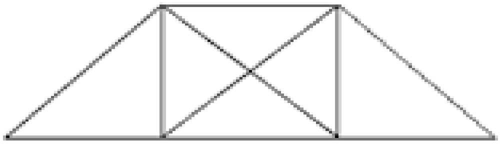
r = 2 $\quad$ 6 = 6

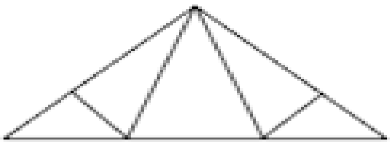
It is a Perfect Truss

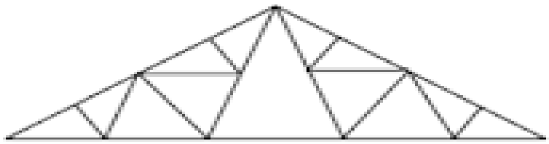
But Unstable (r <3)

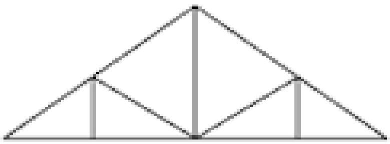
5.4 COMMON TYPES OF ROOF TRUSSES

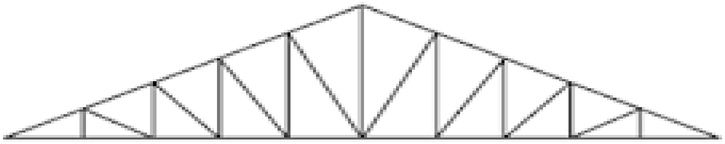

(a) King Post Roof Truss

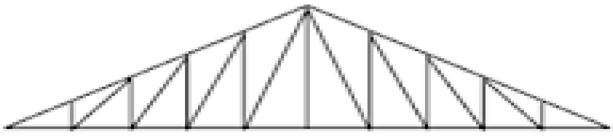

(b) Queen Post Roof Truss

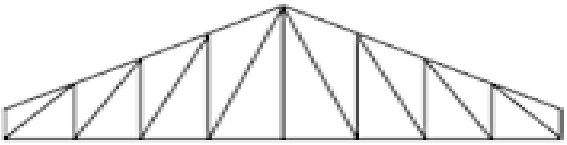

(c) Fink Roof Truss (Short Span)

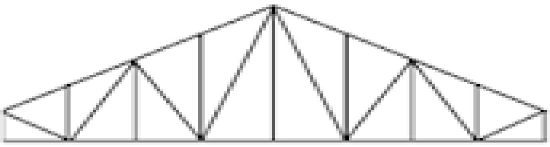

(d) Fink Roof Truss (Long Span)

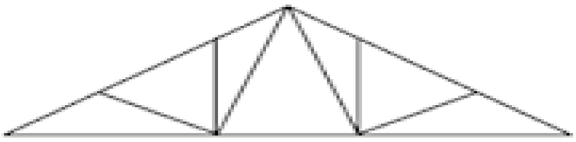

(e) Howe Truss (Short Span)

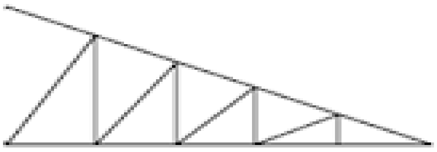

(f) Howe Truss (Long Span)


(g) Pratt Truss


(h) Modified Pratt Truss


(i) Warren Truss


(j) Fan Truss


(k) North Light Roof Truss

5.5 ANALYSIS OF PERFECT FRAME

5.5.1 ASSUMPTIONS MADE IN THE ANALYSIS OF PERFECT FRAME

- The joints of the truss are made with frictionless pins
- The loads are applied at the joints of the truss
- The members of the truss carry axial forces only
- Self-weight of the members is neglected
- All the members of truss are of two force members, along the axes, collinear, equal & opposite

5.5.2 ANALYSIS

Analysis of perfect truss consists of

1. Finding reactions at the supports
2. Finding the internal forces in the members of the truss

REACTIONS OF SUPPORTS

The reactions at the supports are determined by the conditions that the applied force system and the induced support reactions form a system of equilibrium. Treating the whole structure in equilibrium, the support reactions are determined by applying the equilibrium equations $\mathbf{\Sigma H = 0, \Sigma V = 0, \Sigma M = 0}$

INTERNAL FORCES IN THE MEMBERS

The internal forces in the members are determined by the condition that every joint or part of the truss is in equilibrium. Because, if the truss as a whole is in equilibrium, then each part or joint should also be in equilibrium and hence the forces on any part or joint will form a system of equilibrium.

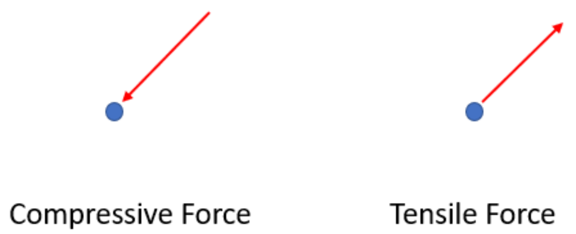
The forces in the members may be found by either analytical or graphical method. The following are the important analytical methods,

- Method of Joints
- Method of Sections
- Method of Tension Co-efficient

5.5.3 NATURE OF MEMBER FORCE

The force in the member of a truss will be either Tensile or Compressive. It is an axial force, assumed to coincide with the axis of the member. Each member of a truss is a two-force member of collinear, equal, and opposite, one at each end, to keep the member in equilibrium.

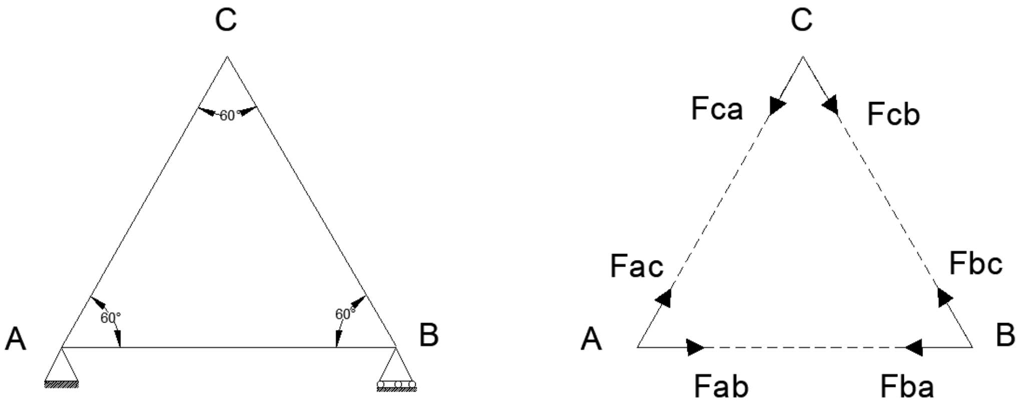
If the force is directed towards the joint (Push) is called **Compressive force** and if it is directed away from the joint (Pull) is called **Tensile force**



2.6 METHOD OF JOINTS

In this method, equilibrium of each joint is considered. The force system at any joint is coplanar concurrent. Hence the equilibrium equations **ΣH = 0 & ΣV = 0** are applied to solve the unknown forces in the free body diagram of a joint. The step-by-step procedure of the method is described below.

1. Determine the support reactions by treating the whole truss as a free body and applying the equations of equilibrium
2. Nature of the member forces are initially assumed as tensile and the free body diagram of the joints of the truss are drawn



For the given figure, if all the members forces are assumed as tensile, then the directions of the member forces can be taken as shown

F_{AB} = Member force at the joint A , directed towards the joint B

F_{BA} = Member force at the joint B, directed towards the joint A

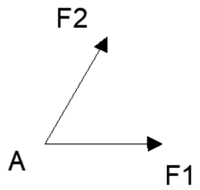
3. Treating each joint in equilibrium, free body diagram of the joints.
4. The coplanar concurrent forces at each joint are in equilibrium, hence the equations of equilibrium **ΣH = 0 & ΣV = 0** are applied to find the unknown member forces.
5. While solving if we get negative value, it shows that the assumed direction of that particular force is wrong, it actually acts in the opposite direction, i.e., its nature is compressive.
6. Free body diagrams of the joints are taken one by one and the magnitude and nature of all the member forces are determined as explained above.
7. Finally results are tabulated

NOTE: As we have only two equations to solve, only 2 unknown forces can be solved at any joint

5.6.1 SPECIAL CONDITIONS

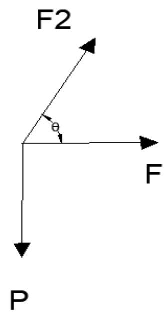
TWO MEMBERS JOINT

- If two members are meeting at a joint, which are not collinear and there is no external force at that joint, then the forces in both the members are zero



$$\mathbf{F_1 \quad = \quad F_2 \quad = \quad 0}$$

- If two members are meeting at a joint which are not collinear and there is a vertical load P as shown the vertical component of inclined force is equal to the vertical load but acts in the opposite direction



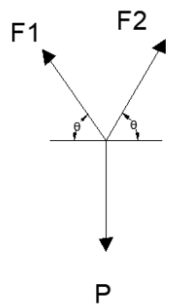
$$\mathbf{F_2 \sin \theta \quad = \quad P}$$

$$\mathbf{F_2 \quad = \quad P/ \sin \theta}$$

Horizontal force F_1 is equal to the horizontal component of the inclined force but acts in the opposite direction.

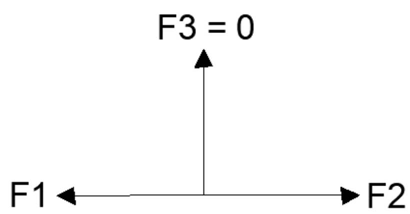
$$\mathbf{F_1 \quad = \quad F_2 \sin \theta}$$

- If two members are meeting at a joint with a vertical external load P as shown, then sum of the vertical components of F_1 and F_2 is equal to P and also the horizontal components of F_1 and F_2 are equal and opposite

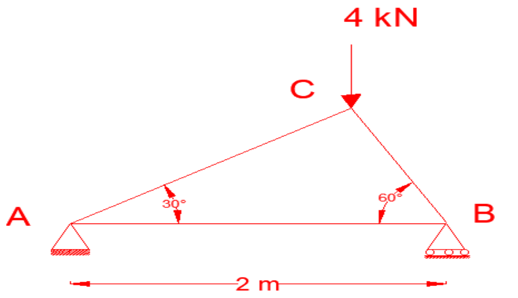


THREE MEMBERS JOINT

If three members are meeting at a joint of which two are collinear, third member being at any angle and if there is no external load at the joint, then the force in the third member is zero.

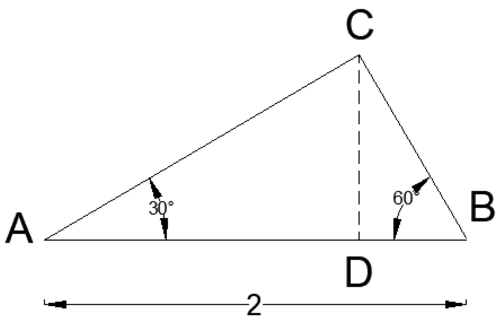


2. Determine the magnitude and nature of forces in all members of the truss by method of joints.



Solution:

STEP 1 : GEOMETRICAL CONFIGURATION



In Δ ABC

$\sin 60^0 = \frac{AC}{AB} = \frac{AC}{2}$

AC = 2 * $\sin 60^0$

AC = 1.732 m

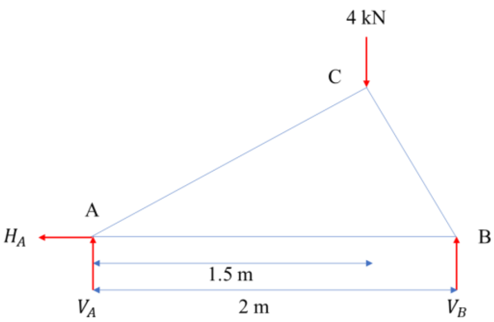
In Δ ACD

$\cos 30^0 = \frac{AD}{AC} = \frac{AD}{1.732}$

AD =1.732 * $\cos 30^0$

AD = 1.5 m

STEP 2 : DETERMINATION OF SUPPORT REACTIONS



ΣH = 0

$H_A = 0$

ΣV = 0

$V_A - 4 + V_B = 0$

$V_A + V_B = 4$

ΣM= 0

$(V_A*0) + (4*1.5) - (V_B*2) = 0$

$6 - 2V_B = 0$

$2V_B = 6$

$\mathbf{V_B = 3\ kN}$

Substitute V_B = 3 kN

$V_A + V_B = 4$

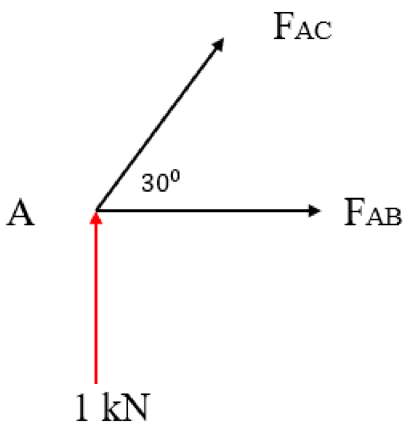
$V_A + 3 = 4$

$V_A = 4 - 3$

$\mathbf{V_A = 1\ kN}$

STEP 3 : DETERMINATION OF MEMBER FORCES

JOINT A



ΣH = 0

$$F_{AB} + F_{AC} \cos 30^0 = 0$$
$$F_{AB} + 0.866\, F_{AC} = 0$$

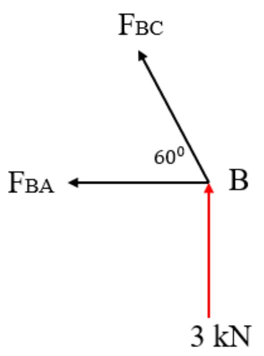
ΣV = 0

$$1 + F_{AC} \sin 30^0 = 0$$
$$F_{AC} \sin 30^0 = -1$$
$$F_{AC} = \frac{-1}{\sin 30^0}$$
$$\mathbf{F_{AC} = -2\ kN}$$

Substitute F_{AC} = 0.5 kN

$$F_{AB} + 0.866\, F_{AC} = 0$$
$$F_{AB} + 0.866 * (-2) = 0$$
$$F_{AB} - 1.732 = 0$$
$$\mathbf{F_{AB} = 1.732\ kN}$$

JOINT B

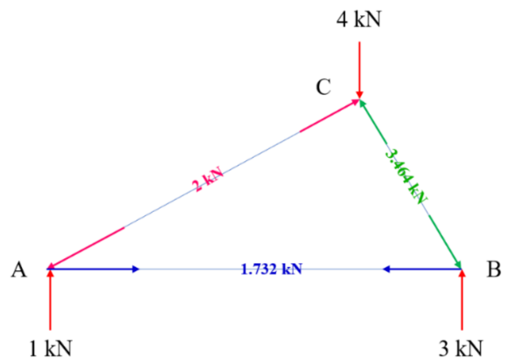


ΣV = 0

$$3 + F_{BC} \sin 60^0 = 0$$
$$F_{BC} \sin 60^0 = -3$$
$$F_{BC} = \frac{-3}{\sin 60^0}$$
$$\mathbf{F_{BC} = -3.464\ kN}$$

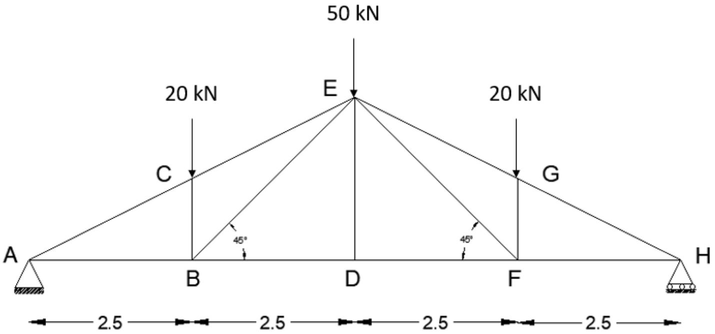
Since the values of F_{AC} and F_{BC} are negative values
The direction of the forces F_{AC} and F_{BC} should be reversed

STEP 4 : RESULT



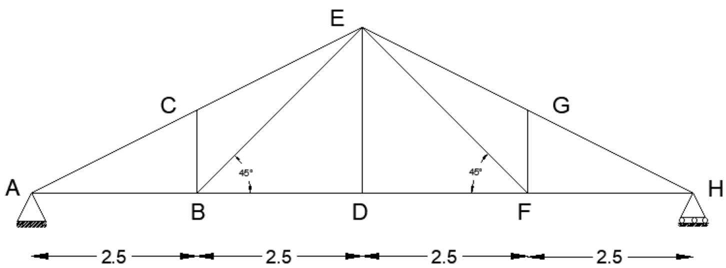
MEMBER		MAGNITUDE	NATURE
AC	=	2 kN	Compression
AB	=	1.732 kN	Tension
CB	=	3.46 kN	Compression

3. Determine the magnitude and nature of forces in all members of the truss. Dimensions are in meter. Use method of Joints.



Solution:

STEP 1 : GEOMETRICAL CONFIGURATION



In Δ BED

$$\tan 45^0 = \frac{ED}{BD} = \frac{ED}{2.5}$$

$$ED = 2.5 * \tan 45^0$$

$$ED = 2.5 \text{ m}$$

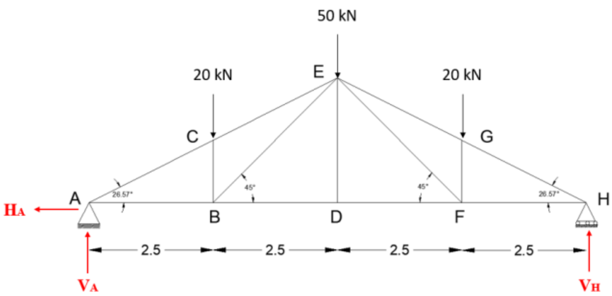
In Δ AED

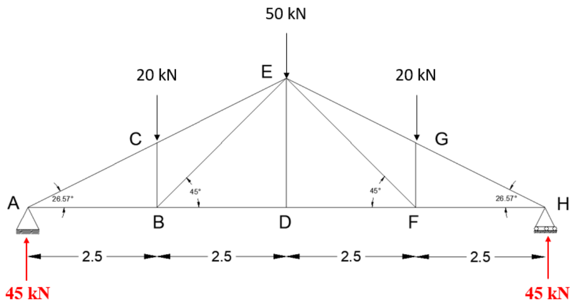
$$\tan\theta = \frac{ED}{AD} = \frac{2.5}{5} = 0.5$$

$$\theta = \tan^{-1}(0.5)$$

$$\theta = 26.57^0$$

STEP 2 : DETERMINATION OF SUPPORT REACTIONS





ΣH = 0

$H_A = 0$

ΣV = 0

$$V_A - 20 - 50 - 20 + V_H = 0$$

$$V_A + V_H = 90$$

ΣM= 0

$$(V_A*0) + (20*2.5) + (50*5) + (20*7.5) - (V_H*10) = 0$$

$$50 + 250 + 150 - 10V_H = 0$$

$$10V_H = 450$$

$$V_H = 45 \text{ kN}$$

Substitute V_H = 45 kN

$$V_A + V_H = 90$$

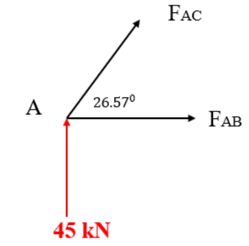
$$V_A + 45 = 90$$

$$V_A = 90-45$$

$$V_A = 45 \text{ kN}$$

STEP 3 : DETERMINATION OF MEMBER FORCES

JOINT A

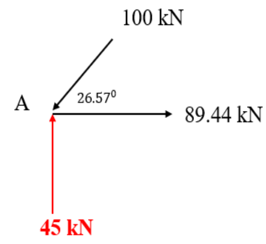


ΣV = 0

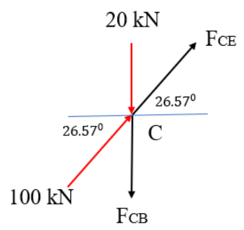
$$45 + F_{AC} \sin 26.57^0 = 0$$
$$F_{AC} \sin 26.57^0 = -45$$
$$F_{AC} = \frac{-45}{Sin\ 26.57^0}$$
$$\mathbf{F_{AC} = -100\ kN}$$

ΣH = 0

$$F_{AB} + F_{AC} \cos 26.57^0 = 0$$
$$F_{AB} + (-100)(\cos 26.57^0) = 0$$
$$\mathbf{F_{AB} = 89.44\ kN}$$



JOINT C

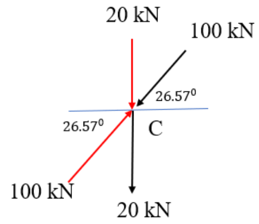


ΣH = 0

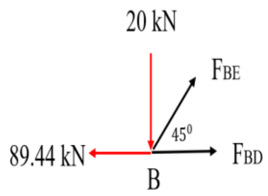
$$100 \cos 26.57^0 + F_{CE} \cos 26.57^0 = 0$$
$$89.44 + F_{CE} \cos 26.57^0 = 0$$
$$F_{CE} \cos 26.57^0 = -89.44$$
$$F_{CE} = \frac{-89.44}{cos\ 26.57^0}$$
$$\mathbf{F_{CE} = -100\ kN}$$

ΣV = 0

$$100 \sin 26.57^0 - 20 + F_{CE} \sin 26.57^0 - F_{CB} = 0$$
$$44.73 - 20 - 44.73 - F_{CB} = 0$$
$$\mathbf{F_{CB} = 20\ kN}$$



JOINT B

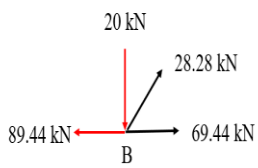


ΣV = 0

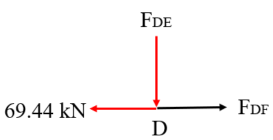
$$-20 + F_{BE} \sin 45^0 = 0$$
$$F_{BE} \sin 45^0 = 20$$
$$F_{BE} = \frac{20}{sin\ 45^0}$$
$$\mathbf{F_{BE} = 28.28\ kN}$$

ΣH = 0

$$-89.44 + F_{BD} + F_{BE} \cos 45^0 = 0$$
$$-89.44 + F_{BD} + 28.28 \cos 45^0 = 0$$
$$-89.44 + F_{BD} + 20 = 0$$
$$\mathbf{F_{BD} = 69.44\ kN}$$



JOINT D

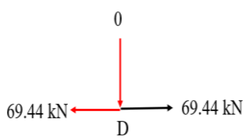


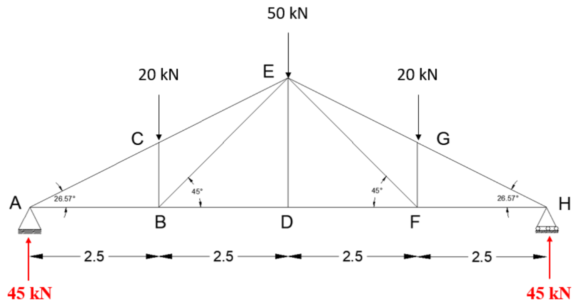
ΣV = 0

$$\mathbf{F_{DE} = 0}$$

ΣH = 0

$$-69.44 + F_{DF} = 0$$
$$\mathbf{F_{DF} = 69.44\ kN}$$



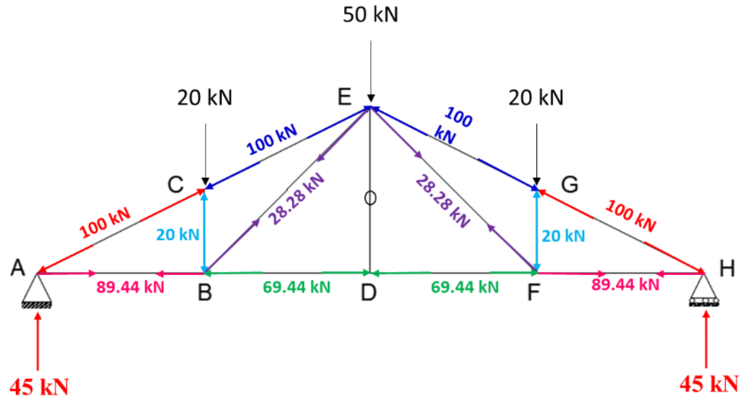


Since the given structure is symmetrical with respect to loading and geometry about y-axis.
The vertical reaction at A and H are equal

Similarly due to symmetry, the forces in the members are also equal in nature and magnitude.

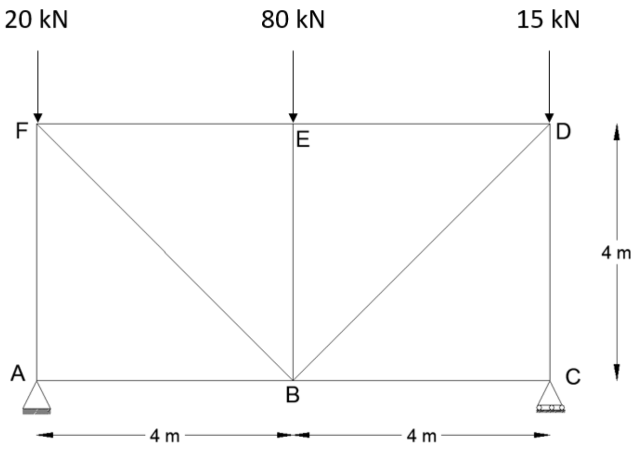
F_{AB}	=	F_{HF}	=	89.44 kN	TENSION
F_{BD}	=	F_{FD}	=	69.44 kN	COMPRESSION
F_{AC}	=	F_{HG}	=	100 kN	COMPRESSION
F_{CE}	=	F_{GE}	=	100 kN	COMPRESSION
F_{BC}	=	F_{GF}	=	20 kN	COMPRESSION
F_{BE}	=	F_{FE}	=	28.28 kN	TENSION

STEP 4 : RESULT



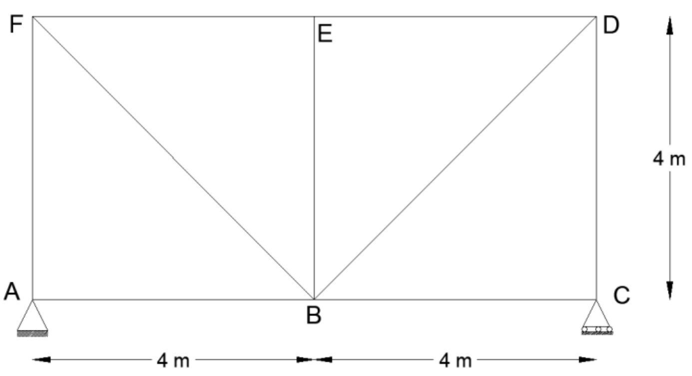
MEMBER				MAGNITUDE	NATURE
AB	=	HF	=	89.44 kN	TENSION
BD	=	FD	=	69.44 kN	COMPRESSION
AC	=	HG	=	100 kN	COMPRESSION
CE	=	GE	=	100 kN	COMPRESSION
BC	=	GF	=	20 kN	COMPRESSION
BE	=	FE	=	28.28 kN	TENSION
		ED	=	0	

4. Determine the magnitude and nature of forces in all members of the truss by method of joints.



Solution:

STEP 1 : GEOMETRICAL CONFIGURATION



In Δ ABF

$$\tan\theta = \frac{AF}{AB} = \frac{4}{4} = 1$$

$$\theta = \tan^{-1}(1)$$

$$\theta = 45^0$$

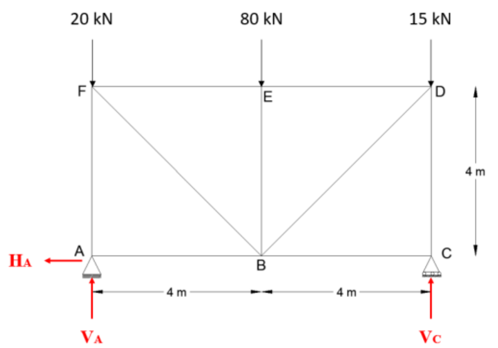
In Δ BCD

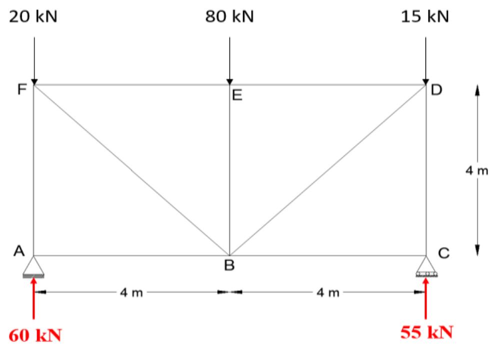
$$\tan\theta = \frac{DC}{BC} = \frac{4}{4} = 1$$

$$\theta = \tan^{-1}(1)$$

$$\theta = 45^0$$

STEP 2 : DETERMINATION OF SUPPORT REACTIONS





ΣH = 0

$H_A = 0$

ΣV = 0

$V_A - 20 - 80 - 15 + V_C = 0$

$V_A + V_C = 115$

ΣM= 0

$(V_A*0) + (20*0) + (80*4) + (15*8) - (V_C*8) = 0$

$320 + 120 - 8V_C = 0$

$8V_C = 440$

$V_C = 55$ kN

Substitute $V_B = 55$ kN

$V_A + V_C = 115$

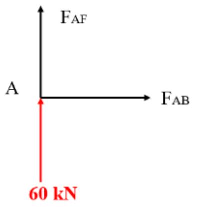
$V_A + 55 = 115$

$V_A = 115-55$

$V_A = 60$ kN

STEP 3 : DETERMINATION OF MEMBER FORCES

JOINT A



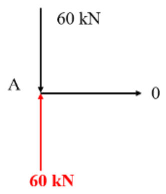
ΣV = 0

$-60 - F_{AF} = 0$

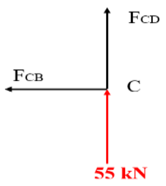
$\mathbf{F_{AF} = -60\ kN}$

ΣH = 0

$\mathbf{F_{AB} = 0}$



JOINT C



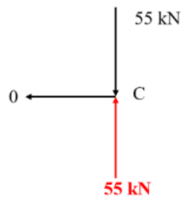
ΣV = 0

$-55 - F_{CD} = 0$

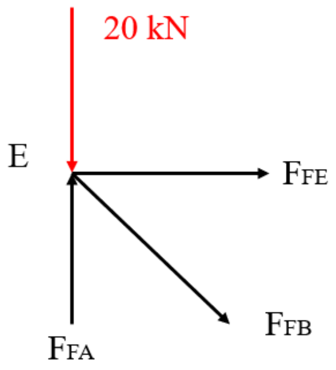
$\mathbf{F_{CD} = -55\ kN}$

ΣH = 0

$\mathbf{F_{CB} = 0}$



JOINT E



ΣV = 0

$20 - F_{FA} - F_{FB} \sin 45^0 = 0$

$20 - 60 - F_{FB} \sin 45^0 = 0$

$F_{FB} \sin 45^0 = 40$

$F_{FB} = \frac{40}{\sin 45^0}$

$\mathbf{F_{FB} = 56.56\ kN}$

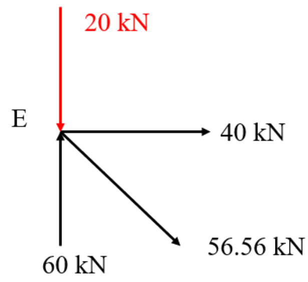
ΣH = 0

$F_{FE} + F_{FB} \sin 45^0 = 0$

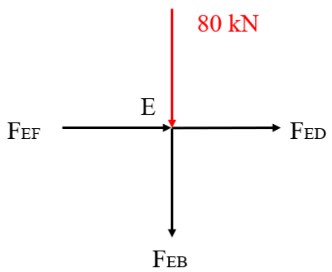
$F_{FE} + (56.56) \sin 45^0 = 0$

$F_{FE} + 40 = 40$

$\mathbf{F_{FE} = -40\ kN}$



JOINT E



ΣV = 0

$-80 - F_{EB} = 0$

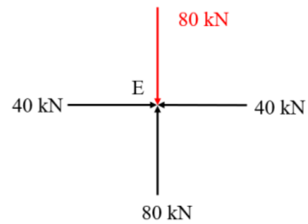
$\mathbf{F_{EB} = -80\ kN}$

ΣH = 0

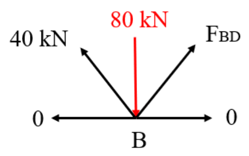
$F_{EF} + F_{ED} = \mathbf{0}$

$40 + F_{ED} = 0$

$\mathbf{F_{ED} = -40\ kN}$



JOINT B

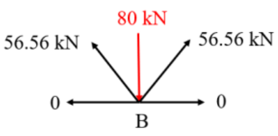


$\Sigma V = 0$ $\quad - 56.56 \sin 45^0 + F_{BD} \sin 45^0 = 0$

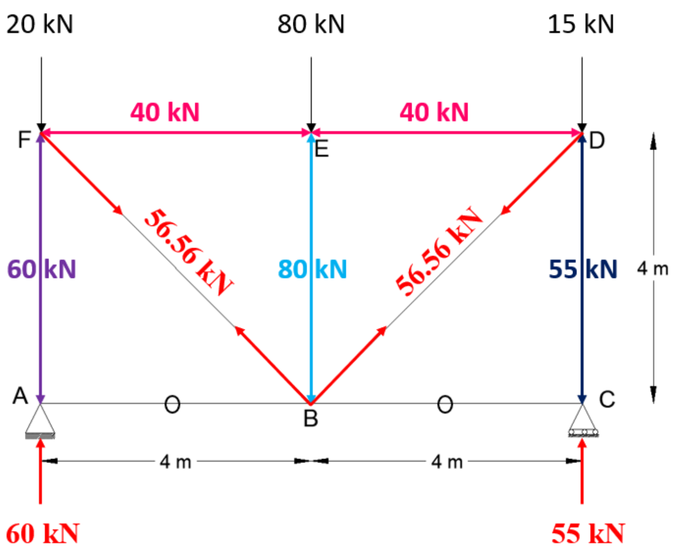
$F_{BD} \sin 45^0 \quad - 56.56 \sin 45^0$

$\Sigma H = 0$ $\quad F_{BD} = \frac{56.56 \sin 45^0}{\sin 45^0}$

$F_{DE} = 56.56 \text{ kN}$

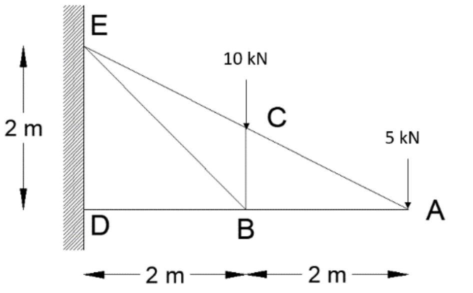


STEP 4 : RESULT



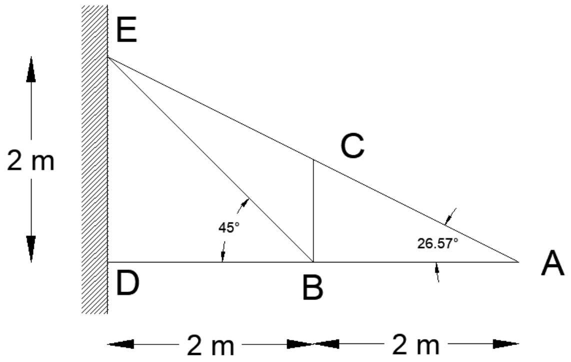
MEMBER		MAGNITUDE	NATURE
AF	=	60 kN	Compression
AB	=	0	-
BC	=	0	-
CD	=	55 kN	Compression
DE	=	40 kN	Compression
FE	=	40 kN	Compression
FB	=	56.56 kN	Tension
BD	=	56.56 kN	Tension
EB	=	80 kN	Compression

5. Determine the magnitude and nature of forces in all members of the truss by method of joints.



Solution:

STEP 1 : GEOMETRICAL CONFIGURATION



In Δ EBD

$$\tan\theta = \frac{ED}{DB} = \frac{2}{2} = 1$$

$$\theta = \tan^{-1}(1)$$

$$\theta = 45^0$$

In Δ EDA

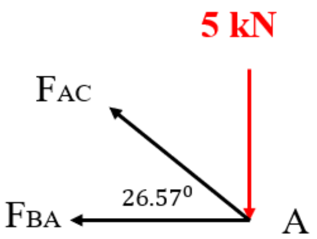
$$\tan\theta = \frac{ED}{AD} = \frac{2}{4} = 0.5$$

$$\theta = \tan^{-1}(0.5)$$

$$\theta = 26.56^0$$

STEP 2 : DETERMINATION OF MEMBER FORCES

JOINT A



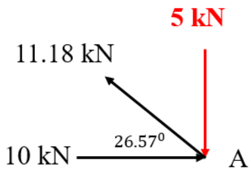
ΣV = 0

$$5 - F_{AC} \sin 26.57^0 = 0$$

$$F_{AC} \sin 26.57^0 = 5$$

$$F_{AC} = \frac{5}{\sin 26.57^0}$$

$$\mathbf{F_{AC} = 11.18\ kN}$$



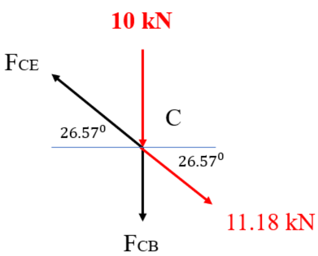
ΣH = 0

$$-F_{BA} - F_{AC} \cos 26.57^0 = 0$$

$$-F_{BA} - 11.18 \cos 26.57^0 = 0$$

$$\mathbf{F_{BA} = -10\ kN}$$

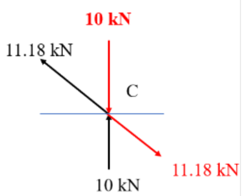
JOINT C



ΣH = 0

$$-F_{CE} \cos 26.57^0 + 11.18 \cos 26.57^0 = 0$$

$$\mathbf{F_{CE} = 11.18\ kN}$$



ΣV = 0

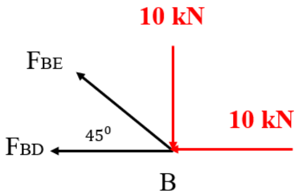
$$-10 - F_{CB} + F_{CE} \cos 26.57^0 - 11 \cos 26.57^0 = 0$$

$$-10 - F_{CB} + 11.18 \cos 26.57^0 - 11.18 \cos 26.57^0 = 0$$

$$-10 - F_{CB} = 0$$

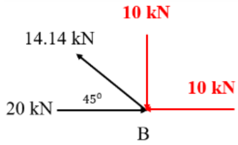
$$\mathbf{F_{CB} = -10\ kN}$$

JOINT B



ΣV = 0

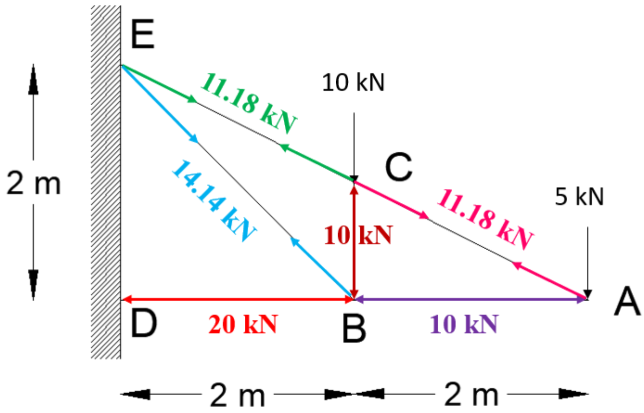
$$10 - F_{BE} \sin 45^0 = 0$$
$$F_{BE} \sin 45^0 = 10$$
$$F_{BE} = \frac{10}{\sin 45^0}$$
$$F_{BE} = 14.14 \text{ kN}$$



ΣH = 0

$$-F_{BD} - F_{BE} \cos 45^0 - 10 = 0$$
$$-F_{BD} - (14.14) \cos 45^0 - 10 = 0$$
$$-F_{BD} - 10 - 10 = 0$$
$$F_{BD} = -20 \text{ kN}$$

STEP 3 : RESULT



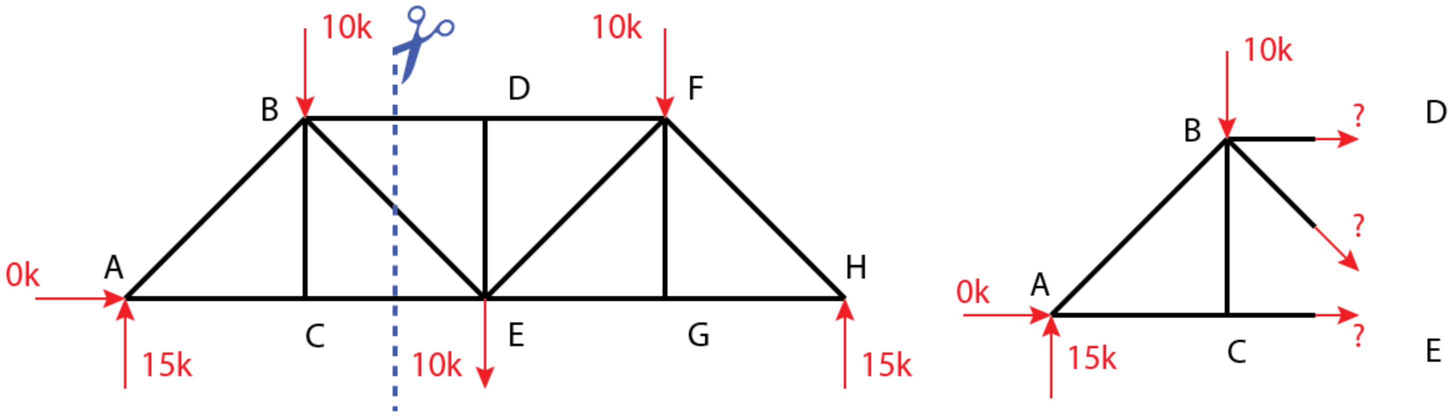
MEMBER		MAGNITUDE	NATURE
AB	=	10 kN	Compression
AC	=	11.18 kN	Tension
BC	=	10 kN	Compression
BD	=	20 kN	Compression
BE	=	14.14 kN	Tension
CE	=	11.18 kN	Tension

5.7 METHOD OF SECTIONS or METHOD OF MOMENTS

Method of sections is an alternative method to find the magnitude and nature of member forces of a statically determinate structure. Method of joints is suitable for a simple truss and when the forces in all members of the truss are to be found out. But if the force in a particular member or forces in few members of a truss are required, method of sections can be effectively used.

In method of sections, a section line is drawn through the members on which the forces are required and the truss is cut into two parts. Then each part of the truss is treated as a free body in equilibrium under the action of external forces on that part and the forces in the members cut by the section line.

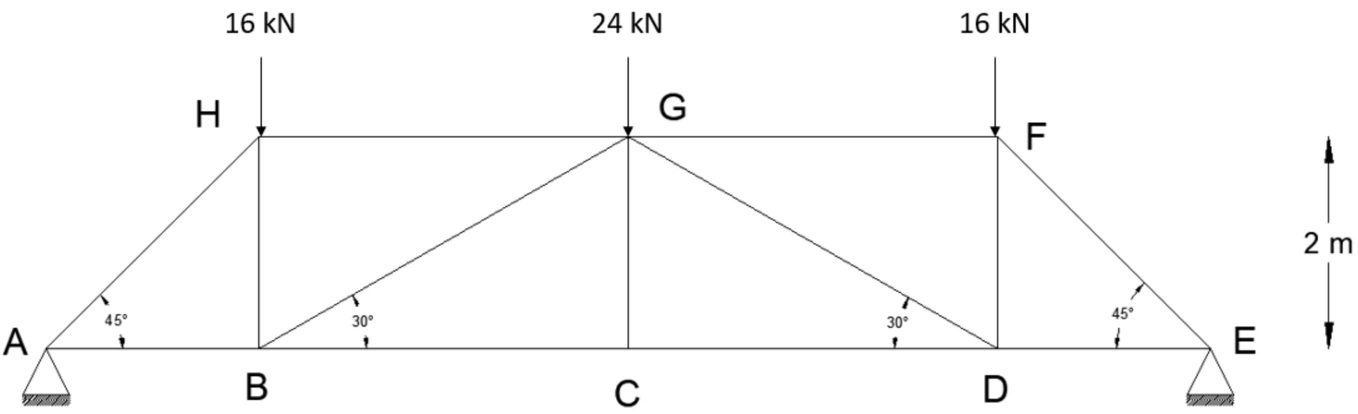
<u>**NOTE:** The section should not pass through more than three unknown member forces</u>



5.8 COMPARISON BETWEEN METHOD OF JOINTS AND METHOD OF SECTIONS

S.No	Point of Compression	Method of Joints	Method of Sections
1	Free body	A joint of the truss	A part / section of the truss
2	Force system on Free body	Coplanar Concurrent	Coplanar Noncurrent and Non-parallel
3	Application of Static equilibrium equations	**ΣH = 0 & ΣV = 0**	**ΣH = 0, ΣV = 0 & ΣM = 0**
4	Number of unknowns that can be determined in Freebody diagram	Two	Three
5	Suitability	When forces in all the members of the truss are required	When forces in any/few members are required

6. Determine the magnitude and nature of forces in all members of the truss by Method of Sections.



Solution:

STEP 1 : GEOMETRICAL CONFIGURATION

In Δ ABF and Δ EFD

$$\tan 45^0 = \frac{HB}{AB}$$

$$AB = \frac{HB}{\tan 45^0} = \frac{2}{1}$$

$AB = 2\ m = ED$

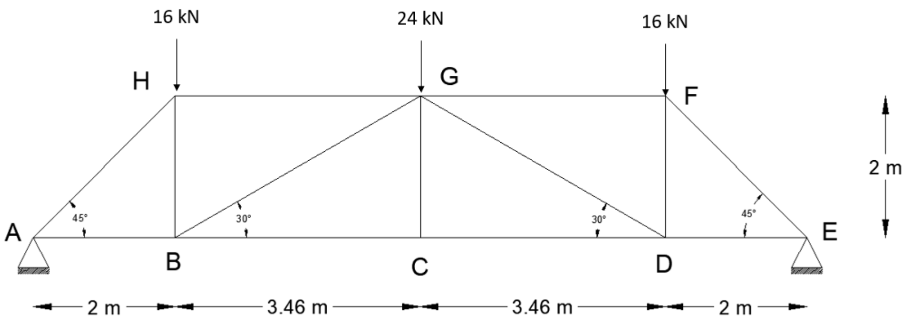
In Δ BCG and Δ DCG

$$\tan 30^0 = \frac{CG}{BC}$$

$$BC = \frac{CG}{\tan 30^0} = \frac{2}{0.577}$$

$BC = 3.46\ m = CD$

$HG = GF = 3.46\ m$

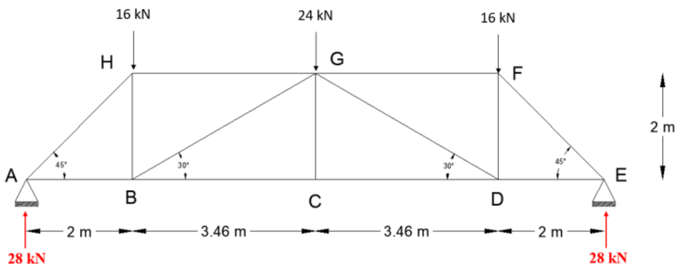


STEP 2 : DETERMINATION OF SUPPORT REACTIONS

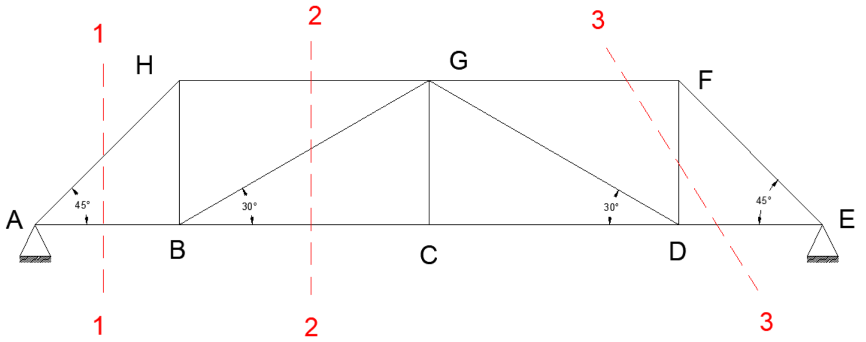
Since the structure is symmetrical about Y-axis Then the support reactions will be half of the total load

$$V_A = V_E = \frac{Total\ Load}{2} = \frac{16+24+16}{2} = \frac{56}{2}$$

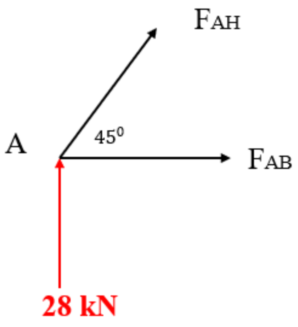
$V_A = V_E = 28\ kN$



STEP 3 : DETERMINATION OF MEMBER FORCES



SECTION 1 - 1



ΣV = 0

$$-28 - F_{AH} = 0$$

$$\mathbf{F_{AH} = -28\ kN}$$

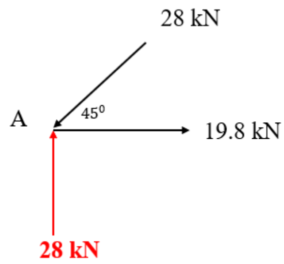
ΣH = 0

$$F_{AH} \cos 45^0 + F_{AB} = 0$$

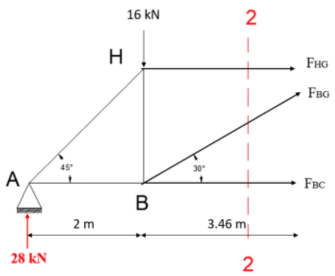
$$(-28) \cos 45^0 + F_{AB} = 0$$

$$F_{AB} = 28 \cos 45^0$$

$$\mathbf{F_{AB} = 19.8\ kN}$$



SECTION 2 - 2



ΣV = 0

$$-28 + 16 - F_{BG} \sin 30^0 = 0$$

$$F_{BG} \sin 30^0 = -12$$

$$F_{BG} = \frac{-12}{\sin 30^0}$$

$$\mathbf{F_{BG} = -24\ kN}$$

$\Sigma M_G = 0$

$$(28*5.46) - (16*3.46) - (F_{BC} * 2) = 0$$

$$152.88 - 55.36 - 2\,F_{BC} = 0$$

$$2\,F_{BC} = 97.52$$

$$\mathbf{F_{BC} = 48.76\ kN}$$

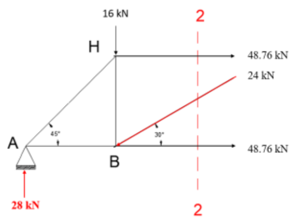
$\Sigma M_C = 0$

$$(28*5.46) - (16*3.46) - (F_{HG} * 2) = 0$$

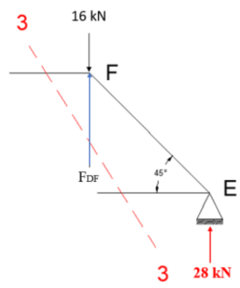
$$152.88 - 55.36 - 2\,F_{HG} = 0$$

$$2\,F_{HG} = 97.52$$

$$\mathbf{F_{HG} = 48.76\ kN}$$



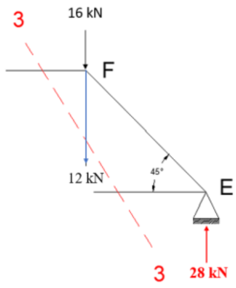
SECTION 3 - 3



ΣV = 0

$$16 - F_{DF} - 28 = 0$$

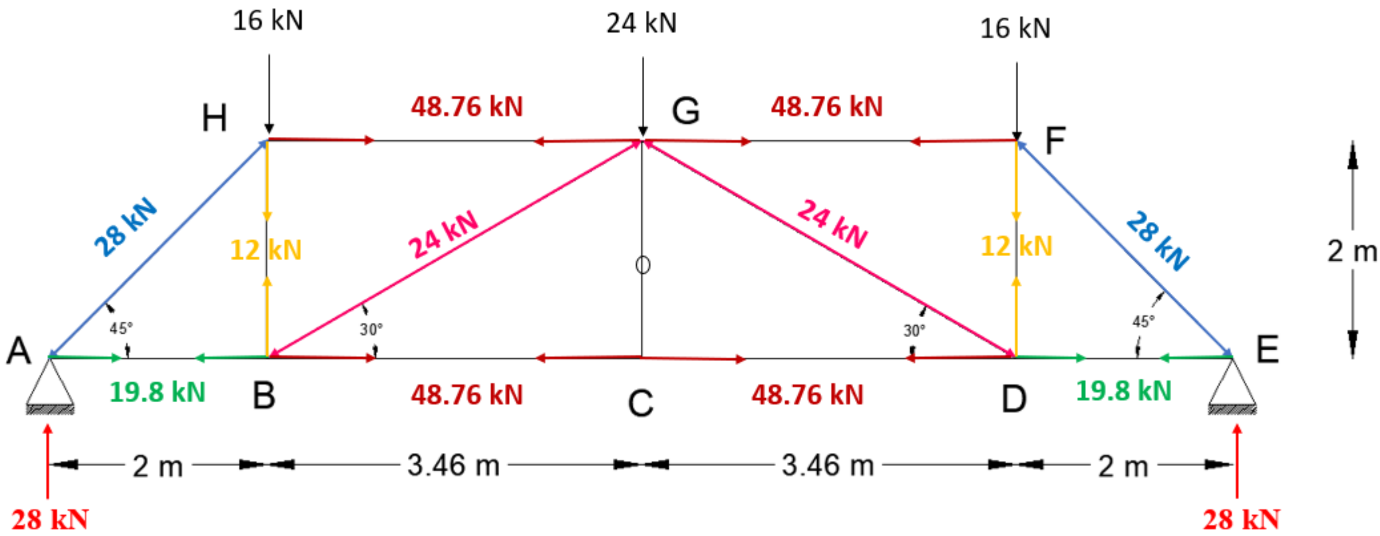
$$F_{DF} = -12 \text{ kN}$$



By Special condition of Three-member principle

$$F_{GC} = 0$$

STEP 4 : RESULT



MEMBER				MAGNITUDE	NATURE
AH	=	EF	=	28 kN	COMPRESSION
AB	=	ED	=	19.8 kN	TENSION
BC	=	DC	=	48.76 kN	TENSION
HG	=	FG	=	48.76 kN	TENSION
HB	=	FD	=	12 kN	TENSION
BG	=	DG	=	24 kN	COMPRESSION
		CG	=	0	-

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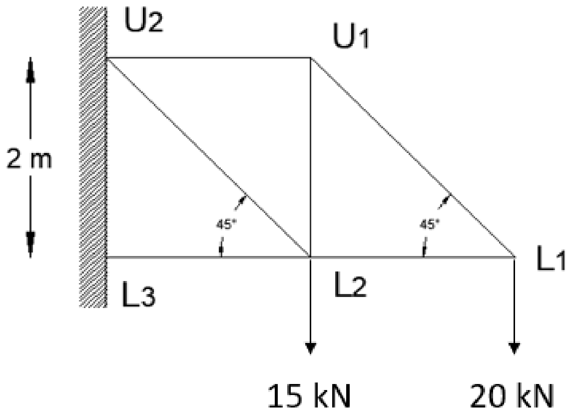
7. **Determine the magnitude and nature of forces in all members of the truss by Method of Sections.**



Solution:

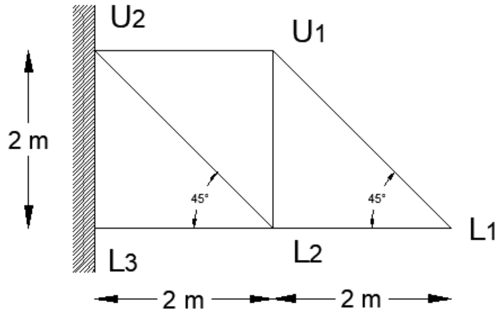
STEP 1 : GEOMETRICAL CONFIGURATION

In Δ $L_2L_3U_2$ and $L_1L_2U_1$

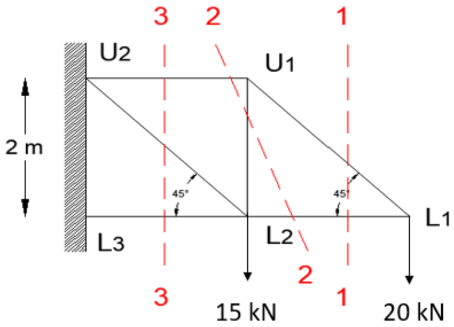
$$\tan 45^0 = \frac{L_2L_3}{L_3U_2} = \frac{L_2L_3}{2}$$

$$L_2L_3 = \tan 45^0 * 2$$

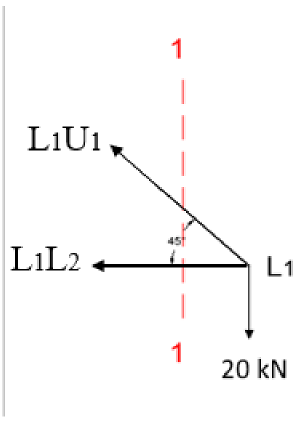
$$L_2L_3 = 2\,m = L_2U_1$$



STEP 2 : DETERMINATION OF MEMBER FORCES



SECTION 1 - 1

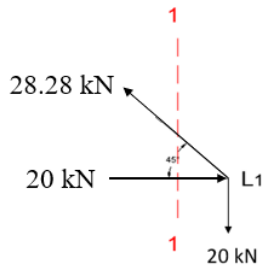


ΣV = 0

$$-20 + L_1U_1 \sin 45^0 = 0$$

$$0.707\, L_1U_1 = 20$$

$$\mathbf{L_1U_1 = 28.28\ kN}$$



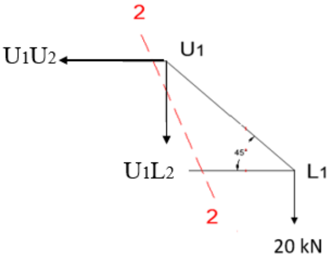
ΣH = 0

$$-L_1L_2 - L_1U_1 \cos 45^0 = 0$$

$$-L_1L_2 - (28.28 * 0.707) = 0$$

$$\mathbf{L_1L_2 = -20\ kN}$$

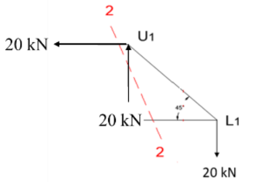
SSECTION 2 - 2



ΣV = 0

$$-20 - U_1L_2 = 0$$

$$\mathbf{U_1L_2 = -20\ kN}$$



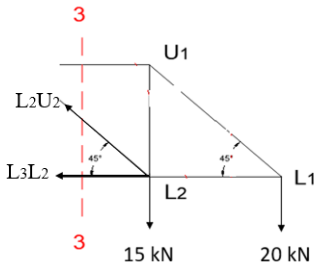
$\Sigma M_{L2} = 0$

$$(20*2) - (2* U_1U_2) = 0$$

$$2U_1U_2 = 40$$

$$\mathbf{U_1U_2 = 20\ kN}$$

SECTION 3 - 3

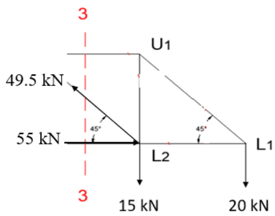


ΣV = 0

$$15 + 20 - L_2U_2 \sin 45^0 = 0$$

$$0.707\ L_2U_2 = 35$$

$$\mathbf{L_2U_2 = 49.5\ kN}$$



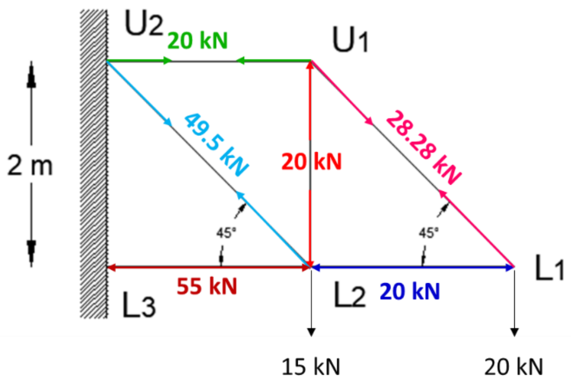
$\Sigma M_{U2} = 0$

$$(20*4) + (15*2) + (2* L_2L_3) = 0$$

$$2\ L_2L_3 = -110$$

$$\mathbf{L_2L_3 = -55\ kN}$$

STEP 4 : RESULT



MEMBER		MAGNITUDE	NATURE
L1L2	=	20 kN	COMPRESSION
L1U1	=	28.28 kN	TENSION
L2U1	=	20 kN	COMPRESSION
L2L3	=	55 kN	COMPRESSION
L2U2	=	49.5 kN	TENSION
U1U2	=	20 kN	TENSION