ANALYSIS AND DESIGN OF A MULTI-STOREY BUILDING USING STAAD.pro

Names and Enrol. Nos. -

Devang Chauhan	(071624)
Jay Purandare	(071620)
Puneet Kumar	(071626)
Shriya Sharma	(071561)
Siddharth Guleria	(061621)

Name of supervisor - Mr. Anil Dhiman

MAY 2011

Submitted in partial fulfillment of the Degree of

Bachelor of Technology

in Civil Engineering

DEPARTMENT OF CIVIL ENGINEERING
JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY
WAKNAGHAT

TABLE OF CONTENTS

CERTIFICATE

This is to certify that the work titled, "Analysis and Design of a Multi-storey Building using Staad.Pro" submitted by Devang Chauhan (071624), Jay Purandare (071620), Puneet Kumar (071626), Shriya Sharma (071561), Siddharth Guleria (061621) has been carried out under my supervision. This work has not been submitted partially or wholly to any other University or Institute for award of this or any other degree or diploma.

Supervisor

Mr. Anil Dhiman, Sr. Lecturer

Civil Engineering Department

Jaypee University of Information Technology

Certified the above mentioned project work has been carried out by the said group of students.

26/5/11

(Dr. Ashok Gupta)

Head of Department

Jaypee University of Information Technology

Solan, Waknaghat

CANDIDATE'S DECLARATION

We hereby certify that the work which is being presented in this report, "Analysis and Design of a Multi-storey Building using STAAD.pro" in partial fulfillment of the requirement for the award of B.Tech degree, submitted in the Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat is an authentic record of our own work carried out from July 2010 to May 2011 under the guidance of Mr. Anil Dhiman, Senior Lecturer in Civil Engineering Department. We have not submitted the matter embodied in the report for the award of any other degree.

(Puneet Kumar)	(Jay Purandare)	(Devang Chauhan)	(Shriya Sharma)	(Siddharth Guleria)
071626	071620	071624	071561	061621

ACKNOWLEDGEMENT

The satisfaction that accompanied the successful completion of any task would be incomplete mentioning the people who made it possible and whose guidance and encouragement served as an oasis in the desert and crowned our efforts with success.

It is a great pleasure and moment of immense satisfaction for us to express our profound gratitude to our mentor **Mr. Anil Dhiman** whose kind cooperation, dynamic guidance, active involvement and constant motivation in the proceedings in the project right from the beginning, has greatly helped us in realizing our study project. Working under his guidance has been an opportunity for us to learn more and more.

We are also grateful to our Head of Department **Dr. Ashok Gupta** whose boosting attitude and incredible co-operation was a great support to the perseverance we had put in during our project.

We feel short of words to express thanks to all faculty of Civil Engineering Department, our seniors and Batch fellows, who have contributed directly or indirectly in this project to complete the dissertation work successfully.

Finally, by no means least, we owe special thanks and indebt to our respective family members for the moral support and affection. Their blessings are the most important strength, which always lighted our path towards success.

This section is a vote of thanks and gratitude towards all those persons who have directly or indirectly contributed in their own special way towards the completion of this project.

\- Project Team

LIST OF ABBREVIATIONS

Symbol	Meaning
DL	Dead load
LL	Live Load
EL	Live Load
WL	Wind load
A_{st}	Area of tension reinforcement
f_y	Characteristic strength of steel
f_{ck}	Characteristic compressive strength of concrete
α_x, α_y	Bending moment coefficient for 2-way slab
l_x	Length of shorter side of slab
l_y	Length of longer side of slab
P_u	Axial load on compression member
d'	Clear cover
A_{sc}	Area of compression reinforcement or area of longitudinal Reinforcement
Q_{ult}	Axial load carrying capacity of the base
C	Cohesion
A_b	Cross-sectional area of base
α	Reduction factor
A_s	Surface area of pile shaft
e_x	Eccentricity
E	Young modulus
I	Moment of inertia
Z	Seismic zone factor
R	Response reduction factor
α_h	Horizontal seismic coefficient
g	Acceleration due to gravity

Chapter 1

Introduction to High Rise Buildings

1.1 INTRODUCTION

A **tower block** or **high-rise** is a tall building or structure used as a residential or office building. Also referred as MDU or 'Multi Dwelling Unit'. High-rise buildings became possible with the invention of the elevator (lift) and cheaper, more abundant building materials. The materials used for the structural system of high-rise buildings are reinforced concrete and steel. Most skyscrapers have a steel frame, while residential blocks are usually constructed out of concrete. High-rise structures pose following design challenges for structural and geotechnical engineers:

Particularly if situated in a seismically active region or if the underlying soils have geotechnical risk factors such as high compressibility or bay mud. They also pose serious challenges to firefighters during emergencies in high-rise structures. New and old building design, building systems like the building standpipe system, HVAC systems (Heating, Ventilation and Air conditioning), fire sprinkler system and other things like stairwell and elevator evacuations pose significant problems.

Apartment blocks have technical and economic advantages in areas with high population density. They have become a distinguished form of housing accommodation in virtually all densely populated urban areas around the world. In contrast with low-rise and single-family houses, apartment blocks accommodate more inhabitants per unit of area of land they occupy and also decrease the cost of municipal infrastructure.

The high-rise building is generally considered as one that is taller than the maximum height which people are willing to walk up; it thus requires mechanical vertical transportation. This includes a rather limited range of building uses, primarily residential apartments, hotels, and office buildings, though occasionally including retail and educational facilities.

1.2 HIGH-RISE BUILDING

HIGH-RISE BUILDING is defined as a building 35 meters or greater in height, which is divided at regular intervals into acceptable levels. To be considered a high-rise building, a structure must be based on solid ground, and fabricated along its full height through deliberate processes (as opposed to naturally-occurring formations).

A high-rise building is distinguished from other tall man-made structures by the following guidelines:

- It must be divided into multiple levels of at least 2 meters height.
- It has fewer than 12 such internal levels, and then the highest undivided portion must not exceed 50% of the total height.
- Indistinct divisions of levels such as stairways shall not be considered floors for purposes of eligibility in this definition.
- Any method of structural support, which is consistent with this definition, is allowable, whether masonry, concrete, or metal frame.
- In the few cases where such a building is not structurally self-supporting (e.g., resting on a slope or braced against a cliff), it may still be considered a high-rise building but is not eligible for any height records unless the record stipulates inclusions of this type.

1.2.1 DEMANDS FOR HIGH RISE LIVING

Following factors are responsible for demand of tall buildings:

1. Scarcity of land in urban areas
2. Increasing demand for business and residential
3. Space
4. Economic growth
5. Technological advancements
6. Innovations in Structural Systems
7. Desire for aesthetics in urban settings
8. Concept of city skyline
9. Cultural significance and prestige
10. Human aspiration to build higher

1.2.2 MINIMUM HEIGHT

- The cutoff between high-rise and low-rise buildings is 35 meters.
- This height was chosen based on an original 12-floor cutoff, used for the following reasons:
  - Twelve floors is normally the minimum height needed to achieve the physical presence, which earns the name high-rise.
  - The twelve-floor limit represents a compromise between ambition and manageability for a worldwide database.
  - A building of fewer floors may only be included as a high-rise when its exact height is known. In most cases, a city is considered to have a satisfactory listing of high-rise buildings when all twelve-floor buildings are counted.

1.3 OBJECTIVE OF THE PROJECT

- Objective is to analyze and design of a 15-storeyed building lying in seismic zone-IV.
- The building will be divided into portal frames and these frames have to be analyzed using the STAAD.PRO software.
- The 15-storeyed portal will be analyzed for dead load, live load, wind load, load and earthquake load combinations.
- The analysis will give the forces arising in the members, namely – transverse beams and columns, due to the above loads and these members were designed for the several forces obtained due to the load combinations.

  1. Slabs
  2. Beams
  3. Columns

- The members will be designed by the Limit State method, according to the guidelines prescribed by IS – 456 : 2000.
- IS-Codes which are to be used are as follows:-

- IS-875:1987 PART-1 for dead load
- IS-875:1987 PART-2 for live load
- IS-875:1987 PART-3 for wind load
- IS-1893:2002 PART-2 for earthquake loads
- IS-456-2000 for limit state design
- IS-13920-1993 for R/F Ductile detailing

1.4 STAGES OF DESIGNING

The approach for designing the proposed building consisted of the following stages.

Estimation of Loads:

For the four-storey building, the analysis was performed and the design done for the following loads:

- Dead load
- Live load
- Earthquake load
- Wind load

The dead load was worked out by assuming a certain thickness for the slab and then the actual thickness was accordingly provided after calculating the required value. The load due to the flooring – screed, finishes, tiles etc. was given due consideration and an allowance was made for future erection of partitions.

The live loads considered were due to the imposed loads in case of educational buildings, as per the specifications of The National Building Code.

Due to increased emphasis being laid on the design of earthquake resistant structures nowadays, the earthquake forces were estimated with the help of the provisions of the revised Seismic Code (IS:1893). The proposed building would lie in Zone IV. The value of the importance factor assigned to the entire structure was 1.5. The load was initially applied to the slabs and through trapezoidal distribution it was transmitted to the columns via beams (longitudinal and transverse), and consequently to the foundations.

The designing was done after analyzing the structure for the above-mentioned loads – individually and for different load combinations recommended in the code.

Analysis of the Structure

The building was divided into portal frames and these frames have been analyzed using the STAAD.pro 2000 software. The analysis gave the forces arising in the members, namely – transverse beams and columns, due to the above loads and these members were designed for the severest of forces obtained due to the load combinations.

Member Design

The members were designed by the Limit State method, according to the guidelines prescribed by IS: 456-2000. For the purpose of design, Design Aids to IS: 456 (SP:16) was also referred to.

Drawing Details

The architectural drawings enabled the understanding of the layout of the building and gave the locations of the various members. The structural drawings were prepared after designing the individual members, showing the details of the reinforcement to be provided. The analysis, designs, and the drawings were compiled in the end in the form of this project report.

Chapter 2

Structural Elements and Loads

2.1 STRUCTURAL ELEMENTS

2.1.1 STEEL CONCRETE COMPOSITE BEAMS

RCC beams are cast in cement concrete reinforced with steel bars. Beams take up compressive and add rigidity to the structure.

- Beams generally carry vertical gravitational forces but can also be used to carry horizontal loads (i.e., loads due to an earthquake or wind).
- The loads carried by a beam are transferred to columns, walls, or girders, which then transfer the force to adjacent structural compression members. In Light frame construction the joists rest on the beam.

Beams generally carry vertical gravitational forces but can also be used to carry horizontal loads (i.e., loads due to an earthquake or wind). The loads carried by a beam are transferred to columns, walls, or girders, which then transfer the force to adjacent structural compression members. In light frame construction the joists rest on the beam.

Beams are characterized by their profile (the shape of their cross-section), their length, and their material. In contemporary construction, beams are typically made of steel, reinforced concrete, or wood. One of the most common types of steel beam is the I-beam or wide-flange beam (also known as a "universal beam" or, for stouter sections, a "universal column"). This is commonly used in steel-frame buildings and bridges. Other common beam profiles are the C-channel, the hollow structural section beam, the pipe, and the angle.

2.1.2 STEEL CONCRETE COMPOSITE COLUMNS

A column is a compression member, comprising either a concrete encased hot-rolled steel section or a concrete filled tubular section of hot-rolled steel and is generally used as a load-bearing member in a composite framed structure.

In a composite column, both the steel and concrete would resist the external loading by Interacting together by bond and friction. Supplementary reinforcement in the concrete encasement prevents excessive spalling of concrete both under normal load and fire conditions.

With the use of composite columns along with composite decking and composite beams it is possible to erect high rise structures in an extremely efficient manner.

The ***advantages*** of composite columns are:

- Increased strength for a given cross sectional dimension.
- Increased stiffness, leading to reduced slenderness and increased buckling resistance.
- Good fire resistance in the case of concrete encased columns.
- Corrosion protection in encased columns.

2.1.3 STEEL CONCRETE COMPOSITE SLABS

A **concrete slab** is a common structural element of modern buildings. Horizontal slabs of steel reinforced concrete, typically between 10 and 50 centimeters thick, are most often used to construct floors and ceilings, while thinner slabs are also used for exterior paving.

In high rise buildings and skyscrapers, thinner, pre-cast concrete slabs are slung between the steel frames to form the floors and ceilings on each level.

Reinforcement design of slabs:

- A *one way slab* needs moment resisting reinforcement only in its short-direction. Because, the moment along long axes is so small that it can be neglected. When the ratio of the length of long direction to short direction of a slab is greater than 2 it can be considered as a one way slab.

- A *two way slab* needs moment resisting reinforcement in both directions. If the ratio of the lengths of long and short side is less than one then moment in both directions should be considered in design.

For a suspended slab, there are a number of designs to improve the strength-to-weight ratio. In all cases the top surface remains flat, and the underside is modulated:

- *Corrugated*, usually where the concrete is poured into a corrugated steel tray. This improves strength and prevents the slab bending under its own weight. The corrugations run across the short dimension, from side to side.
- A *ribbed slab*, giving considerable extra strength on one direction.
- A *waffle slab*, giving added strength in both directions.

Slab Construction:

A concrete slab may be prefabricated or in site. Prefabricated concrete slabs are built in a factory and transported to the site, ready to be lowered into place between steel or concrete beams. They may be pre-stressed (in the factory), post-stressed (on site), or unstressed. It is vital that the supporting structure is built to the correct dimensions, or the slabs may not fit.

- In situ concrete slabs are built on the building site using formwork - a type of boxing into which the wet concrete is poured. If the slab is to be reinforced, the rebars are positioned within the formwork before the concrete is poured in. Plastic tipped metal, or plastic bar chairs are used to hold the rebar away from the bottom and sides of the form-work, so that when the concrete sets it completely envelops the reinforcement. For a *ground slab*, the form-work may consist only of sidewalls pushed into the ground. For a *suspended slab*, the form-work is shaped like a tray, often supported by a temporary scaffold until the concrete sets.

2.2 LOADS ON THE STRUCTURES

2. 2.1 DEAD LOADS

These will include the weight of all components at each level, viz., *roof* including water tanks, *Barsatis*, Parapets, roof finishes, slabs, beams, elevator machine room etc. and including all plasters and surface cladding etc., and each *floor level* including fixed masonry or other partitions, infill walls, columns, slabs and beams, weight of stairs, cantilever balconies, parapets and plastering or cladding wherever used.

Typically they are relatively constant throughout the structure's life, and so they are also known as Permanent loads. The designer can also be relatively sure of the magnitude of the load as it is closely linked to density of the material, which has a low variance, and is normally responsible for the specification of the component.

Dead loads also include forces set up by irreversible changes in a structure's constraints. For example, loads due to settlement, the effects of pre-stress or due to shrinkage and creep in concrete.

The unit weights may be taken from IS: 875 (Part 1) or ascertained from the manufacturer.

2.2.2 IMPOSED FLOOR LOADS

IS 875 (Part 2) deals with the imposed loads on roofs, floors, stairs, balconies, etc., for various occupancies.

There is a provision for reduction in the imposed loads for certain situations, e.g. for large span beams and number of storeys above the columns of a storey. The earthquake code IS: 1893 (Part 1)-2002 permits general reduction in roof and floor imposed load when considering the load combination with the earthquake loading.

But the two types of reductions, that is, in IS: 875 (Part 2) and IS: 1893 (Part 1) are *not* to be taken together.

2.2.3 EARTHQUAKE LOADS

An earthquake is a sudden, rapid shaking of the Earth caused by the breaking and shifting of rock beneath the Earth's surface. For hundreds of millions of years, the forces of plate tectonics have shaped the Earth as the huge plates that form the Earth's surface move slowly over, under, and past each other. Sometimes the movement is gradual. At other times, the plates are locked together, unable to release the accumulating energy. When the accumulated energy grows strong enough, the plates break free causing the ground to shake. Most earthquakes occur at the boundaries where the plates meet; however, some earthquakes occur in the middle of plates.

Ground shaking from earthquakes can collapse buildings and bridges; disrupt gas, electric, and phone services; and sometimes trigger landslides, avalanches, flash floods, fires, and huge, destructive ocean waves (tsunamis). Buildings with foundations resting on unconsolidated landfill and other unstable soil, and trailers and homes not tied to their foundations are at risk because they can be shaken off their mountings during an earthquake. When an earthquake occurs in a populated area, it may cause deaths and injuries and extensive property damage.

The dynamic response of building to earthquake ground motion is the most important cause of earthquake-induced damage to buildings. The damage that a building suffers primarily depends not upon its displacement, but upon acceleration. Whereas displacement is the actual distance the ground and building may move during an earthquake, acceleration is a measure of how quickly they change speed as they move. The conventional approach to earthquake resistant design of buildings depends upon providing the building with strength, stiffness and inelastic deformation capacity which are great to withstand a given level of earthquake-generated force. This is generally accomplished through the selection of an appropriate structural configuration and the carefully detailing of structural members, such as beams and columns, and the connections between them.

For working out the earthquake loading on a building frame, the dead load and imposed load and weights are to be lumped at each column top on the basis of contributory areas.

The imposed load is to be reduced as specified in IS: 1893 (Part1)-2002 for seismic load determination.

2.2.4 WIND LOADS

The wind load on a building shall be calculated using IS: 875 Part 3 for:

a) The building as a whole

b) Individual structural elements as roofs and Walls

c) Individual cladding units including glazing and their fixings.

2.3 MATERIALS

2.3.1 CEMENT

Ordinary Portland cement conforming to IS 269 - 1976 shall be used along with fly ash after carrying out the design mix from approved consultant.

2.3.2 REINFORCEMENT

Cold twisted high yield strength deformed bars grade Fe 415 conforming to IS: 1786-1985, or preferably TMT bars of standard manufacturer e.g. TATA Steel, SAIL or equivalent shall be used.

2.3.3 SPECIFICATIONS OF CONCRETE

The following grades of concrete mix may be adopted or as required for safe design:

(a) For RCC columns in lowest few storeys: M35

(b) For RCC columns in the middle few storeys: M30

(c) For RCC columns in the top few storeys: M25

(d) For beams, slabs, staircase etc.: M25

(e) Max. Water cement Ratio: 0.45

(f) Minimum cement content: 300 kg/m^3 of concrete.

(g) Admixtures of approved brand may be used as per mix design

Chapter 3

Problem Formulation

3.1 GENERAL

In this chapter, the buildings data have been described. The loads taken from various parts of IS-875 have been imposed on the building and ultimate loads coming on beams from slabs are calculated. Seismic coefficient method is applied to calculate base-shear due to earthquakes, as per IS-1893:2002. Wind loads are also applied separately as per IS-875-part-III.

3.1.1 FUNDAMENTAL DATA REGARDING THE BUILDING

Designing and analyzing of a twin structure to the BSNL Telephone Exchange Building at JUIT (8-Storey) as a 15-Storey Structure.

- Height of the building ($2.9 \times 15 =$ **43.5** m) *(Hence High rise > 35m)*
- Total Columns at the base : 7 / Columns in the 1st Storey : 14
- Columns in the superstructure (2nd to 15th Storey) : 21
- Slab thickness 0.15 m.

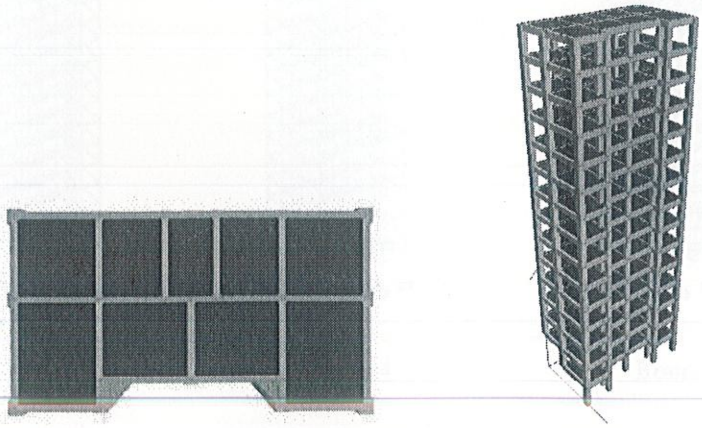


Fig. 3.1: Plan and Perspective view of the building.

3.1.2 GROUPING OF BEAMS (CATEGORIZED ON THE BASIS OF LENGTH)

- Beams 1: 0.60 × 0.30 m
- Beams 2: 0.60 ×0.30 m
- Beams 3: 0.60 × 0.30 m
- Beams 4: 0.60 × 0.30 m
- Beams 5: 0.60 × 0.30 m

Figure 3.2 shows the building with respective beams highlighted.

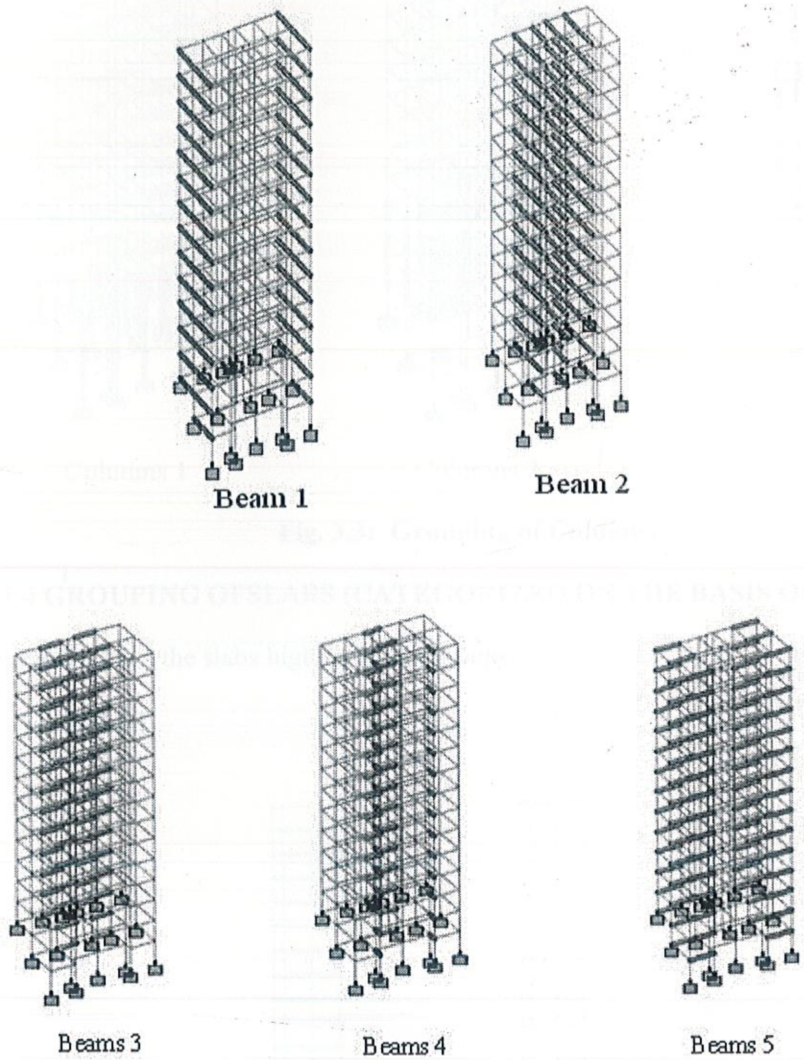
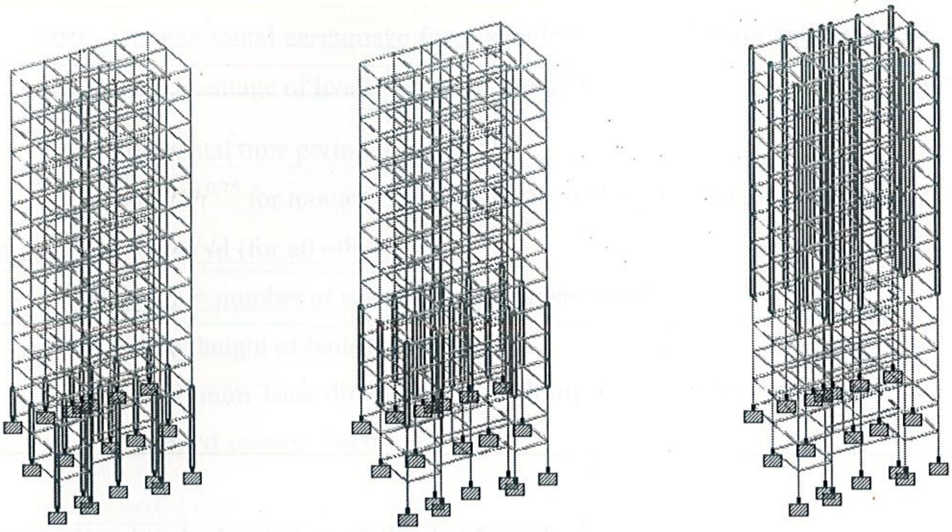


Fig. 3.2: Grouping of Beams.

3.1.3 GROUPING OF COLUMNS (CATEGORIZED ON THE MIX OF CONCRETE USED)

Figure 3.3 shows the columns highlighted in groups.

- Columns 1 : 0.60 × 0.40 m
- Columns 2 : 0.60 × 0.60 m
- Columns 3 : 0.40 × 0.40 m



Columns 1 Columns 2 Column 3

Fig. 3.3: Grouping of Columns.

3.1.4 GROUPING OFSLABS (CATEGORIZED ON THE BASIS OF LOADS)

Figure 2 shows the slabs highlighted in groups.

- Slabs 1
- Slabs 2

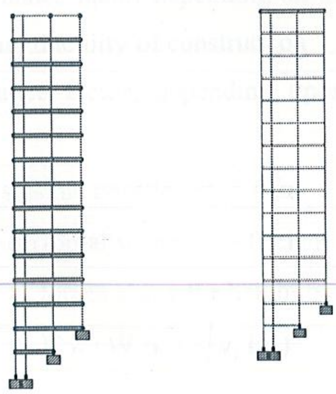


Fig. 3.4: Grouping of Slabs.

3.2 ANALYSIS AND DESIGN PHILOSOPHIES USED

3.2.1 SEISMIC COEFFICIENT METHOD

Following is the procedure used for calculating the equivalent lateral loads on buildings using seismic coefficient method as per IS-1893-2002 (Part-I).

i) India has been divided into four zones with regard to horizontal seismic coefficients. For important structures these coefficient can be increased by 50%. The horizontal earthquake force should be calculated for full dead load and some percentage of live loads as given below

ii) The fundamental time period is given by:

$T = 0.075h^{0.75}$ for moment resisting frame without bracing or shear walls,

$T = 0.09h/\sqrt{d}$ (for all others)

Where, n = number of storeys including basement

H = total height of buildings in m.

d = maximum base dimension of building in m, in direction parallel to applied seismic force.

The base shear is calculated by the following formula:

$$V_B = KC\alpha_h W$$

where,

α_h = design seismic coefficient $= \beta I \alpha_0$

W = total dead load and appropriate percentage of live load

C = a coefficient that depends on the fundamental time period

T = fundamental time period in seconds

K = performance factor depending on the structural framing system and brittleness and ductility of construction

I = Importance factor, depending upon the life and function of the structure

α_h = design seismic coefficient $= \beta I \alpha_0$

α_0 = basic horizontal seismic coefficient

Distribution of forces along the height of building is given by

$$Q_i = V_B (W_i h_i^2 / \sum w_i h_i^2)$$

Where

Q_i = lateral forces at the floor i

V_B = base shear

W_i = load of the floor i

h_i = height measured from the base of the building to the floor i

n = number of storeys including the basement.

3.2.2 LOAD COMBINATIONS

Structural design of various members has to be done by Limit State Method, as per IS 456-2000 for which the following load combinations should be used to work out the maximum member forces:-

Using

DL for DEAD LOAD

LL for LIVE LOAD

EQX for SEISMIC LOAD (X) DIRECTION

EQZ for SEISMIC LOAD (Z) DIRECTION

WLX for WIND LOAD (X) DIRECTION

WLZ for WIND LOAD (Z) DIRECTION

- 7 : 1.5(D.L+L.L)
- 8 : 1.2(D.L+L.L+E.L.X)
- 9 : 1.2(D.L+L.L-E.L.X)
- 10 : 1.2(D.L+L.L+E.L.Z)
- 11 : 1.2(D.L+L.L-E.L.Z)
- 12 : 1.5(D.L+E.L.X)
- 13 : 1.5(D.L+E.L.Z)
- 14 : 1.5(D.L-E.L.X)
- 15 : 1.5(D.L-E.L.Z)
- 16 : 0.9D.L+1.5E.L.X
- 17 : .9 D.L-1.5E.L.X
- 18 : .9D.L+1.5E.L.Z
- 19 : .9D.L-1.5E.L.Z
- 20 : 1.2(D.L+L.L+W.L.X)
- 21 : 1.2(D.L+L.L-W.L.X)
- 22 : 1.2(D.L+L.L+W.L.Z)
- 23 : 1.2(D.L+L.L-W.L.Z)
- 24 : 1.5(D.L+W.L.X)
- 25 : 1.5(D.L-W.L.X)
- 26 : 1.5(D.L+W.L.Z)
- 27 : 1.5(D.L-W.L.Z)
- 28 : 0.9D.L+1.5W.L.X
- 29 : 0.9D.L-1.5W.L.X
- 30 : 0.9D.L+1.5W.L.Z
- 31 : 0.9D.L-1.5W.L.Z

3.3 LIMIT STATE METHOD

- A structure with appropriate degrees of reliability should be able to withstand safely all loads that are liable to act on it throughout its life and it should also be able to satisfy the serviceability requirements, i.e. it will not reach a limit state
- Limit state concept takes into account the probabilistic and structural variation in the material properties, loads and safety factors.

3.3.1 WHAT ARE THE LIMIT STATES?

- Limit states are the acceptable limits for the safety and serviceability requirements of the structure before failure occurs.
- The design of structures by this method will thus ensure that they will not reach limit states and will not become unfit for the use for which they are intended.
- It is worth mentioning that structures will not just fail or collapse by violating (exceeding) the limit states. Failure, therefore, implies that clearly defined limit states of structural usefulness has been exceeded.

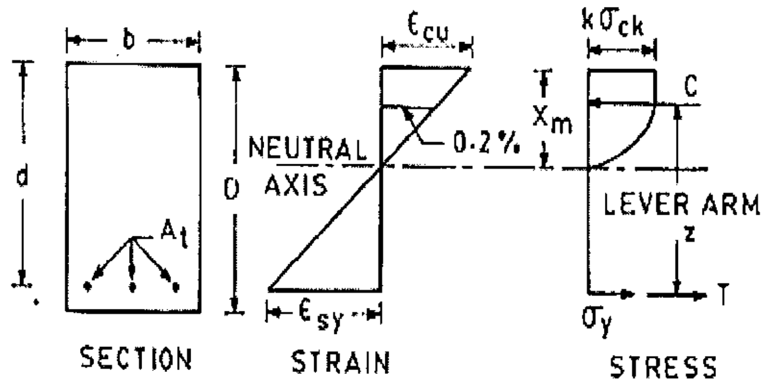


Fig: 3.5 Stress strain Curve in Limit State Design

3.3.2 ASSUMPTIONS IN LIMIT STATE METHOD

- Plane sections normal to the axis remain plane after bending.
- This assumption ensures that the cross-section of the member does not warp due to the loads applied.
- It further means that the strain at any point on the cross-section is directly proportional to its distance from the neutral axis
- The maximum strain in concrete at the outer most compression fibre is taken as 0.0035 in bending.
- This is a clearly defined limiting strain of concrete in bending compression beyond which the concrete will be taken as reaching the state of collapse.
- It is very clear that the specified limiting strain of 0.0035 does not depend on the strength of concrete.
- The acceptable stress-strain curve of concrete is assumed to be parabolic.
- The maximum compressive stress-strain curve in the structure is obtained by reducing the values of the top parabolic curve in two stages.

  a) Dividing by 1.5 due to size effect and secondly, again dividing by 1.5 considering the partial safety factor of the material. The middle and bottom curves represent these stages.

  b) The maximum compressive stress in bending is limited to the constant value of $0.446 f_{ck}$ for the strain ranging from 0.002 to 0.0035

- The tensile strength of concrete is ignored.

  a) Concrete has some tensile strength (very small but not zero). Yet, this tensile strength is ignored and the steel reinforcement is assumed to resist the tensile stress.

  b) However, the tensile strength of concrete is taken into account to check the deflection and crack widths in the limit state of serviceability.

- The maximum strain in the tension reinforcement in the section at failure shall not be less than *fy*/(1.15 *Es*) + 0.002, where *fy* is the characteristic strength of steel and *Es* = modulus of elasticity of steel.

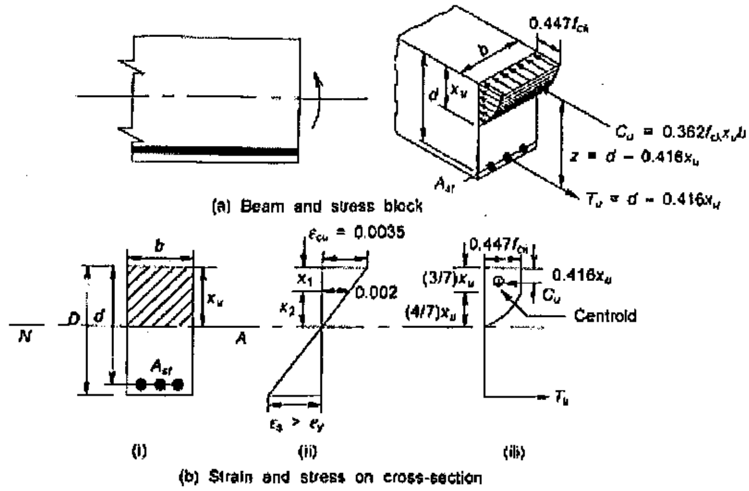


Fig: 3.6 Stress strain on Cross-section.

- This assumption ensures ductile failure in which the tensile reinforcement undergoes a certain degree of inelastic deformation before concrete fails in compression.

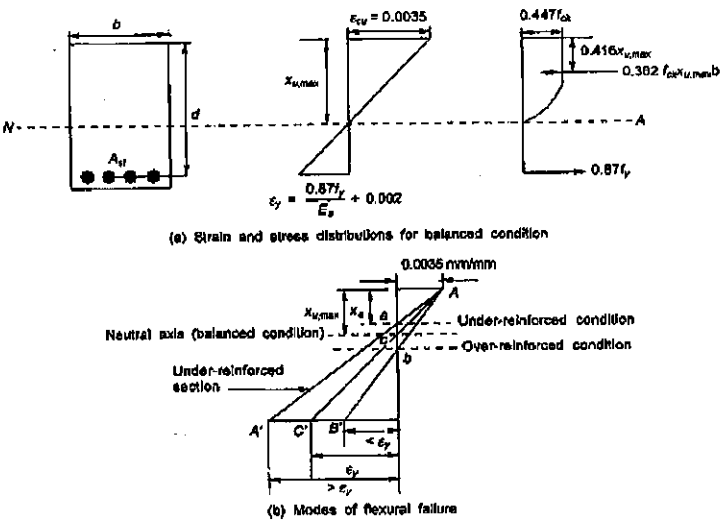


Fig: 3.7 Strain distributions for various modes of flexural failure.

3.4 WORKING STRESS METHOD

- Concrete and steel act together elastically and the relationship between loads and stresses is linear right up to the collapse of the structure
- Permissible stress for concrete and steel are not exceeded anywhere in the structure when it is subjected to the worst combination of working loads
- Bond between steel and concrete is perfect within the elastic limit of steel
- Concrete is elastic i.e. the stress in concrete varies linearly from zero at the neutral axis to a maximum at the extreme fiber
- The modular ratio $m = 280/3\ \sigma_{cb}$ where σ_{cb} is the permissible compressive stress in bending in N/sq mm can be used to determine the stresses in steel and concrete
- The permissible stresses are prescribed by the codes to provide suitable factors of safety to allow for uncertainties in the estimation of working loads and variation in properties of materials
- It uses a factor of safety of 3 w.r.t cube strength for concrete and a FOS of 1.78 w.r.t yield strength for steel

3.4.1 DRAWBACKS OF WORKING STRESS METHOD

- Since the method deals only with the elastic behavior, it neither shows its real strength nor gives the true FOS of the structure against failure
- The inelastic behaviour of concrete starts right from very low stresses and the actual stress distribution in a concrete section cannot be described by a triangular stress diagram.

  Modular ratio itself is an imaginary quantity
- Since FOS is on the stresses under working loads, there is no way to account for different degrees of uncertainty associated with different types of loads
- Modular ratio design results in larger %age of compression steels than while using limit state design
- Because of creep and nonlinear stress-strain relationship, concrete does not have a definite modulus of elasticity as in steel.

Chapter 4

Analysis with STAAD.pro

4.1 INTRODUCTION TO STAAD.PRO

STAAD.Pro is a general purpose program for performing the analysis and design of a wide variety of types of structures. The basic three activities which are to be carried out to achieve that goal - a) model generation b) the calculations to obtain the analytical results c) result verification - are all facilitated by tools contained in the program's graphical environment.

- STAAD.Pro is an analysis and design software for structural engineering.
- STAAD.PRO features a state-of-the-art user interface, visualization tools, powerful analysis and design engines with advanced finite element and dynamic analysis capabilities.
- STAAD.PRO is the professional's choice for the analysis and design of steel, concrete, composite, timber, aluminum and cold-formed steel structures.

4.1.1 LIST OF IS-CODES USED IN STAAD-PRO

- IS 465:2000 Reinforce concrete design (limit state method)
- IS 13920:1993 Ductile Detailing for Earthquake
- IS 1893:2002 Criteria for Seismic Analysis of Structures
- IS 800: Steel Design
- IS 801:1988 Cold Form Steel
- IS:802-1995 (Part 1) Steel Design for Overhead Transmission Line Towers

4.1.2 METHOD USED IN STAAD-PRO

- ANALYSIS
  - "Stiffness Method" also known as the "Displacement Method".
  - Finite element method
    - Plate element
    - Solid element
- DESIGN
  1. Limit state design
  2. Plastic design

4.1.3 STEPS INVOLVED IN STAAD FOR ANALYSIS AND DESIGN

- Modeling is done first by drawing the frame in GUI or by inputting coordinates in the STAAD editor.
- Specification of nodes and members
- Properties are defined and assigned to beams and columns.
- Supports are applied.
- Loads and Load Combinations are defined and assigned to the structure.
- Analysis options and commands are given.
- Concrete design commands are given as per IS-456: 2000 code.
- Analysis is run and results are obtained in the form of tables.

4.1.4 DESIGN STEPS

1. IS-456: 2000 code was chosen for design.
2. Define Grade of Steel and Concrete
3. Define Maximum and Minimum Criteria as Requirement

4.2 ANALYSIS RESULTS

4.2.1 BEAM END FORCES SUMMARY (Table 1)

	Beam	L/C	Node	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Max Fx	105	26 1.5(D.L+	50	**2496.018**	-5.196	-2.788	0.716	0.430	-5.689
Min Fx	112	14 1.5(D.L-E.	57	**-1796.797**	-3.270	-13.793	-0.329	13.443	0.499
Max Fy	130	14 1.5(D.L-E.	62	-197.907	**753.886**	-0.811	5.098	-1.841	-481.352
Min Fy	130	7 1.5(D.L+L.L	62	256.309	**-1007.733**	1.450	-7.044	2.644	642.387
Max Fz	41	31 0.9D.L-1.5	3	1943.459	-7.380	**186.449**	-0.485	-387.930	-9.545
Min Fz	50	26 1.5(D.L+	12	195.787	-5.677	**-195.252**	-0.218	422.265	-10.257
Max Mx	80	27 1.5(D.L-W	28	-2.149	-33.647	-16.804	**31.270**	8.760	-13.890
Min Mx	81	27 1.5(D.L-W	30	-0.528	48.055	15.416	**-29.182**	-8.566	29.728
Max My	50	26 1.5(D.L+	12	195.787	-5.677	-195.252	-0.218	**422.265**	-10.257
Min My	41	31 0.9D.L-1.5	3	1943.459	-7.380	186.449	-0.485	**-387.930**	-9.545
Max Mz	130	7 1.5(D.L+L.L	62	256.309	-1007.733	1.450	-7.044	2.644	**642.387**
Min Mz	130	14 1.5(D.L-E.	62	-197.907	753.886	-0.811	5.098	-1.841	**-481.352**

4.2.2 NODAL DISPLACEMENT SUMMARY (Table 2)

			Horizontal	Vertical	Horizontal	Resultant	Rotational		
	Node	L/C	X ft	Y ft	Z ft	ft	rX rad	rY rad	rZ rad
Max X	285	28 0.9D.L+1.	**0.075**	-0.006	0.022	0.079	0.000	0.000	-0.000
Min X	293	25 1.5(D.L-W	**-0.076**	-0.012	0.029	0.083	0.000	-0.000	0.000
Max Y	304	14 1.5(D.L-E.	0.024	**0.018**	-0.018	0.035	-0.000	0.000	-0.000
Min Y	304	26 1.5(D.L+	-0.001	**-0.026**	0.210	0.212	0.001	-0.000	0.000
Max Z	335	26 1.5(D.L+	-0.001	-0.007	**0.210**	0.210	0.001	-0.000	0.000
Min Z	335	31 0.9D.L-1.5	-0.000	-0.015	**-0.182**	0.182	-0.001	-0.000	0.000
Max rX	325	26 1.5(D.L+	-0.001	-0.002	0.054	0.054	**0.002**	-0.000	0.000
Min rX	325	31 0.9D.L-1.5	-0.000	-0.006	-0.048	0.048	**-0.002**	-0.000	0.000
Max rY	285	24 1.5(D.L+	0.075	-0.012	0.029	0.081	0.000	**0.000**	-0.000
Min rY	285	29 0.9D.L-1.5	-0.076	-0.012	-0.001	0.077	0.000	**-0.000**	0.000
Max rZ	63	7 1.5(D.L+L.L	-0.000	-0.007	0.000	0.007	-0.000	0.000	**0.001**
Min rZ	63	14 1.5(D.L-E.	0.000	0.005	-0.000	0.005	0.000	-0.000	**-0.001**
Max Rs	296	26 1.5(D.L+	-0.001	-0.026	0.210	**0.212**	0.001	0.000	0.000

4.2.3 SUMMARY FOR SUPPORT REACTIONS (Table 3)

			Horizontal	Vertical	Horizontal	Moment		
	Node	L/C	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Max Fx	15	25 1.5(D.L-W	**241.042**	1074.706	-98.520	-57.539	68.376	-119.967
Min Fx	62	7 1.5(D.L+L.L	**-273.098**	2773.805	6.522	14.622	-2.180	-672.032
Max Fy	62	7 1.5(D.L+L.L	-273.098	**2773.805**	6.522	14.622	-2.180	-672.032
Min Fy	62	17 .9 D.L-1.5	207.514	**-2052.308**	-4.988	-10.487	1.483	501.369
Max Fz	3	31 0.9D.L-1.5	41.546	2005.213	**489.211**	318.872	-101.768	-2.789
Min Fz	12	26 1.5(D.L+	49.842	223.422	**-523.833**	-412.712	-114.806	-16.997
Max Mx	12	31 0.9D.L-1.5	-33.238	1997.047	474.063	**318.874**	98.209	-4.436
Min Mx	3	26 1.5(D.L+	-31.733	241.029	-490.854	**-412.756**	107.231	1.371
Max My	3	26 1.5(D.L+	-31.733	241.029	-490.854	-412.756	**107.231**	1.371
Min My	12	26 1.5(D.L+	49.842	223.422	-523.833	-412.712	**-114.806**	-16.997
Max Mz	62	14 1.5(D.L-E.	210.386	-2045.367	-4.917	-10.500	1.511	**504.128**
Min Mz	62	7 1.5(D.L+L.L	-273.098	2773.805	6.522	14.622	-2.180	**-672.032**

Chapter 5

Manual Design of Structural Elements

5.1 DESIGN OF BEAMS

Group 1 (+)

b = 300mm

D = 600mm

$F_y = 415\ N/mm^2$

$F_{ck} = 25\ N/mm^2$

$M_{ulim} = 0.362x_{umax}/d\ (d-0.42x_{umax}/d)\ F_{ck}bd^2 = 0.362 \times 0.48 \times 300 \times 25 \times 540^2(1-0.42 \times 0.48) = 303.40$ kNm

$M_u = 0.362\ f_{ck}\ x_u\,(d - 0.416x_u)$

$250.340 \times 10^6 = 0.362 \times 25 \times x_u \times 300\ (540 - 0.416x_u)$

$92206.26 = x_u\ (540 - 0.416x_u)$

$92206.26 = 540x_u - 0.416x_u^2$

$x_u = 202.27$ mm

$M_u = 0.87f_y\,A_{st}\ (d-0.416x_u)$

$250.340 \times 10^6 = 0.87 \times 415\ A_{st}\ (d-0.416 \times 202.27)$

$A_{st} = 1621.02mm^2$

<u>6 Bars of 18 mm diameter</u>

(-ve B.M.)

b = 300mm

D = 600mm

$F_y = 415\ N/mm^2$

$F_{ck} = 25\ N/mm^2$

$M_{ulim} = 0.362x_{umax}/d\ (d-0.42x_{umax}/d)\ F_{ck}bd^2\ = 0.362 \times 0.48 \times 300 \times 25\times 540^2(1-0.42 \times 0.48)$

$= 303.40$ kNm

$M_u = 0.362\ f_{ck}\ x_u(d-0.416x_u)$

$229.374 \times 10^6 = 0.362 \times 25 \times x_u \times 300\ (540 - 0.416x_u)$

$84483.97 = x_u\ (540 - 0.416x_u)$

$84483.97 = 540x_u - 0.416x_u^2$

$X_u = 181.95$ mm

$M_u = 0.87f_y A_{st}\ (d-0.416x_u)$

$229.374 \times 10^6 = 0.87 \times 415\ A_{st}\ (d-0.416 \times 202.27)$

$A_{st} = 1368.26mm^2$

<u>6 Bars of 18 mm diameter</u>

Group 2

b = 300mm

D = 600mm

$F_y = 415\ N/mm^2$

$F_{ck} = 25\ N/mm^2$

$M_{ulim} = 0.362x_{umax}/d\ (d-0.42x_{umax}/d)\ F_{ck}bd^2\ = 0.362 \times 0.48 \times 300 \times 25\times 540^2(1-0.42 \times 0.48)$

$= 303.40$ kNm

$M_u = 0.362\ f_{ck}\ x_u(d-0.416x_u)$

$182.055 \times 10^6 = 0.362 \times 25 \times x_u \times 300\ (540 - 0.416x_u)$

$670552.24 = x_u\ (540 - 0.416x_u)$

$$670552.24 = 540x_u - 0.416x_u^2$$

$$x_u = 649.03 \text{ mm}$$

$$M_u = 0.87f_y A_{st} (d - 0.416x_u)$$

$$182.055 \times 10^6 = 0.87 \times 415\ A_{st} (d - 0.416 \times 202.27)$$

$$A_{st} = 1867.52 \text{mm}^2$$

6 Bars of 20 mm diameter

b = 300mm

D = 600mm

$F_y = 415 \text{ N/mm}^2$

$F_{ck} = 25 \text{ N/mm}^2$

$M_{ulim} = 0.362x_{umax}/d\ (d - 0.42x_{umax}/d)\ F_{ck}bd^2 = 0.362 \times 0.48 \times 300 \times 25 \times 540^2(1 - 0.42 \times 0.48)$

$= 303.40$ kNm

$$M_u = 0.362\ f_{ck}\ x_u (d - 0.416x_u)$$

$$183.495 \times 10^6 = 0.362 \times 25 \times x_u \times 300\ (540 - 0.416 x_u)$$

$$67585.63 = x_u (540 - 0.416x_u)$$

$$67585.63 = 540x_u - 0.416x_u^2$$

$$x_u = 140.32 \text{ mm}$$

$$M_u = 0.87f_y A_{st} (d - 0.416x_u)$$

$$183.495 \times 10^6 = 0.87 \times 415\ A_{st} (d - 0.416 \times 202.27)$$

$$A_{st} = 1055.22 \text{mm}^2$$

4 Bars of 20 mm diameter

Group 3

b = 300mm

D = 600mm

$F_y = 415\ N/mm^2$

$F_{ck} = 25\ N/mm^2$

$M_{ulim} = 0.362x_{umax}/d\ (d\text{-}0.42x_{umax}/d)\ F_{ck}bd^2 = 0.362 \times 0.48 \times 300 \times 25 \times 540^2(1\text{-}0.42 \times 0.48)$

$= 303.40\ kNm$

$M_u = 0.362\ f_{ck}\ x_u\ (d\text{-}0.416x_u)$

$169.768 \times 10^6 = 0.362 \times 25 \times x_u \times 300\ (540 - 0.416x_u)$

$62529.650 = x_u\ (540 - 0.416x_u)$

$62529.650 = 540x_u - 0.416x_u^2$

$x_u = 128.52\ mm$

$M_u = 0.87f_y\ A_{st}\ (d\text{-}0.416x_u)$

$169.768 \times 10^6 = 0.87 \times 415\ A_{st}\ (d\text{-}0.416 \times 202.27)$

$A_{st} = 966.43mm^2$

4 Bars of 20 mm diameter

b = 300mm

D = 600mm

$F_y = 415\ N/mm^2$

$F_{ck} = 25\ N/mm^2$

$M_{ulim} = 0.362x_{umax}/d\ (d\text{-}0.42x_{umax}/d)\ F_{ck}bd^2 = 0.362 \times 0.48 \times 300 \times 25 \times 540^2(1\text{-}0.42 \times 0.48)$

$= 303.40\ kNm$

$M_u = 0.362\ f_{ck}\ x_u\ (d\text{-}0.416x_u)$

$220.421 \times 10^6 = 0.362 \times 25 \times x_u \times 300\ (540 - 0.416x_u)$

$81186.37 = x_u\ (540 - 0.416x_u)$

$81186.37 = 540x_u - 0.416x_u^2$

$x_u = 173.54\ mm$

$M_u = 0.87 f_y A_{st} (d - 0.416 x_u)$

$220.421 \times 10^6 = 0.87 \times 415\ A_{st} (d - 0.416 \times 202.27)$

$A_{st} = 1186.21 mm^2$

<u>4 Bars of 20 mm diameter</u>

Group 4

$b = 300mm$

$D = 600mm$

$F_y = 415\ N/mm^2$

$F_{ck} = 25\ N/mm^2$

$M_{ulim} = 0.362 x_{umax}/d\ (d - 0.42 x_{umax}/d)\ F_{ck} bd^2 = 0.362 \times 0.48 \times 300 \times 25 \times 540^2 (1 - 0.42 \times 0.48)$

$= 303.40\ kNm$

$A_{stlim} = M_{ulim} / 0.87\ f_y (d - 0.42 x_{umax})$

$= 303.40 \times 10^6 / 0.87 \times 415 (d - 0.42 \times 259.2)$

$= 1949.10\ mm^2$

$M_{u2} = M_u - M_{ulim}$

$= 481.352 - 303.40$

$= 177.952$

So, we will design it as doubly reinforced beam.

To find A_{sc}, We need to find out f_{sc} and f_{cc}

$f_{sc} = d'/d = 60/540 = 0.11$

$f_{sc} = 350.8\ N/mm^2$

$f_{cc} = 0.446\ f_{ck} = 0.446 \times 25 = 11.15\ N/mm^2$

$M_{u2} = A_{sc} (f_{sc} - f_{cc}) (d' - d)$

$177.952 \times 10^6 = A_{sc} (350.8 - 11.15) (540 - 60)$

$A_{sc} = 1091.56\ mm^2$

4 bars of 20mm Diameter

$A_{sc}(f_{sc}-f_{cc}) = 0.87\ f_y\ A_{st2}$

$A_{st2} = 1026.86\ mm^2$

$A_{st} = A_{st1} + A_{st2}$

10 Bars of 20 mm Diameter = $2975.96\ mm^2$

CHECK FOR REINFORCEMENT

In tension,

$A_{stmin} = 0.85\ bd/ f_y$

$= 0.85 \times 300 \times 540 / 415$

$= 315.81\ mm^2$

$A_{stmax} = 0.04 \times 300 \times 600 = 7200\ mm^2$

In compression,

$A_{sc} = .2/100 \times 300 \times 600 = 360\ mm^2$

$A_{stmax} = 4/100 \times 300 \times 600 = 7200\ mm^2$

b = 300mm

D = 600mm

$F_y = 415\ N/mm^2$

$F_{ck} = 25\ N/mm^2$

$M_u = 642.387$ kNm

$M_{ulim} = 0.362x_{umax}/d\ (d-0.42x_{umax}/d)\ F_{ck}bd^2 = 0.362 \times 0.48 \times 300 \times 25 \times 540^2(1-0.42 \times 0.48)$

$= 303.40$ kNm

$A_{stlim} = M_{ulim} / 0.87\ f_y\ (d - 0.42 x_{umax})$

$= 303.40 \times 10^6 / 0.87 \times 415\ (d - 0.42 \times 259.2)$

$= 1949.10\ mm^2$

$M_{u2} = M_u - M_{ulim}$

$= 642.387 - 303.40$

$= 338.987$ kNm

So, we will design it as doubly reinforced beam.

To find A_{sc} , We need to find out f_{sc} and f_{cc}

$f_{sc} = d'/d = 60/540 = 0.11$

$f_{sc} = 350.8\ n/mm^2$

$f_{cc} = 0.446\ f_{ck} = 0.446 \times 25 = 11.15\ N/mm^2$

$M_{u2} = A_{sc}\ (f_{sc} - f_{cc})\ (d' - d)$

$338.987 \times 10^6 = A_{sc}\ (350.8 - 11.15)\ (540 - 60)$

$A_{sc} = 2079.27 mm^2$

<u>14 bars of 14mm Diameter</u>

$A_{sc}(f_{sc} - f_{cc}) = 0.87\ f_y\ A_{st2}$

$A_{st2} = 1956.03 mm^2$

$A_{st} = Ast_1 + A_{st2}$

16 Bars of 20 mm Diameter = $3905.13 mm^2$

In tension,

$A_{stmin} = 0.85\ bd / f_y$

$= 0.85 \times 300 \times 540 / 415$

$= 315.81\ mm^2$

$A_{stmax} = 0.04 \times 300 \times 600 = 7200\ mm^2$

In compression,

$A_{sc} = .2/100 \times 300 \times 600 = 360\ mm^2$

$A_{stmax} = 4/100 \times 300 \times 600 = 7200\ mm^2$

Group 5

b = 300mm

D = 600mm

$F_y = 415\ N/mm^2$

$F_{ck} = 25\ N/mm^2$

$M_{ulim} = 0.362 x_{umax}/d\ (d\text{-}0.42 x_{umax}/d)\ F_{ck} bd^2 = 0.362 \times 0.48 \times 300 \times 25 \times 540^2 (1\text{-}0.42 \times 0.48)$

$= 303.40\ kNm$

$$M_u = 0.362\ f_{ck}\ x_u\ (d\text{-}0.416x_u)$$

$$108.035 \times 10^6 = 0.362 \times 25 \times x_u \times 300\ (540 - 0.416x_u)$$

$$39791.90 = x_u\ (540 - 0.416x_u)$$

$$39791.90 = 540x_u - 0.416x_u^2$$

$$x_u = 78.43\ mm$$

$$M_u = 0.87 f_y A_{st}\ (d\text{-}0.416x_u)$$

$$108.035 \times 10^6 = 0.87 \times 415\ A_{st}\ (d\text{-}0.416 \times 202.27)$$

$$A_{st} = 589.75 mm^2$$

<u>2 Bars of 20 mm diameter</u>

b = 300mm

D = 600mm

$F_y = 415\ N/mm^2$

$F_{ck} = 25\ N/mm^2$

$M_{ulim} = 0.362x_{umax}/d\ (d-0.42x_{umax}/d)\ F_{ck}bd^2 = 0.362 \times 0.48 \times 300 \times 25 \times 540^2(1-0.42 \times 0.48)$

$= 303.40$ kNm

$M_u = 0.362\ f_{ck}\ x_{ub}\ (d-0.416x_u)$

$111.659 \times 10^6 = 0.362 \times 25 \times x_u \times 300\ (540 - 0.416x_u)$

$41126.70 = x_u\ (540 - 0.416x_u)$

$41126.70 = 540x_u - 0.416x_u^2$

$x_u = 81.24$ mm

$M_u = 0.87f_y\ A_{st}\ (d-0.416x_u)$

$111.659 \times 10^6 = 0.87 \times 415\ A_{st}\ (d-0.416 \times 202.27)$

$A_{st} = 610.75 mm^2$

<u>2 Bars of 20 mm diameter</u>

5.2 DESIGN OF COLUMNS

$P_u = 2496.018$ kN

$M_z = 242.644$ kN-m

$M_x = 422.265$ kN-m

Fe415, M35

1. Check for accidental Eccentricity

Equivalent of loads given by :

$M_x/p = 422.265 \times 10^3/2496.018$

$= 169.18$ mm

$M_z/p = 24.644 \times 10^3/2496.018$

$= 97.21$ mm

Both are more than 20mm

2. Assume percentage of steel

(Assume steel larger than required by P and M_x)

$d'/D = 60/600 = 0.1$

$M_x/f_{ck}bd^2 = 422.265 \times 10^6 / 35 \times 600 \times 600^2 = 0.06$

$P_u/f_{ck}bD = 2496.018 \times 10^3 / 35 \times 600 \times 600 = 0.20$

$b/f_{ck} = 0.02$

Assume a higher value of $p/f_{ck} = 0.03$

$p = 0.03 \times 35 = 0.05\ \%$

$A_s = 1.05 \times 600 \times 600 / 100 = 3780 mm^2$

Use of 10 – 22mm Φ bars;

$A_s = 3799.4\ mm^2$

3. Find moment capacities M_x and M_y

About x-axis,

$d'/D = 0.1$, $P/f_{ck}\ bD = 0.20$; $P/f_{ck} = 0.03$

$M_{x1}/f_{ck}bd^2 = 0.08$

$M_{x1} = 0.08 \times 35 \times 600 \times 600^2$

$= 604.8$ kN-m

About z-axis,

$d'/D = 0.1$, $P/f_{ck}\ bD = 0.20$; $P/fck = 0.03$

$M_{z1}/f_{ck}bd^2 = 0.08$

$M_{z1} = 0.08 \times 35 \times 600 \times 600^2$ $= 604.8$ kN-m

4. Calculate α_n

$P_z = 0.45\ f_{ck}A_c + 0.75\ A_sf_y$

$= 0.45 \times 35 \times 600 \times 600 + 0.75 \times 415 \times 3780$

$= 6846.52$ kN

$P/P_z = 2496.018 / 6846.52$

$= 0.36$

$\alpha_n = 2/3\ [\ 1 + 5/2 \times 0.36]$

$= 1.27$

5. Criterion for bi-axial bending

$(M_x/M_{x1})^{\alpha n} + (M_z/M_{z1})^{\alpha n} <= 1.0$

$(422.265/604.8)^{1.27} + (242.644/604.8)^{1.27}$

$= 0.63 + 0.31$

$= 0.94$

Which is less than 1. Hence it's safe

DESIGN OF COLUMN 2

$P_u =$ 2250.686 KN

$M_z =$ 124.722 KN-m

$M_x =$ 262.299 KN-m

Fe415, M30 400×400

1. Check for accidental Eccentricity Equivalent of loads given by :

$M_x/p = 262.299 \times 10^3/2250.686$

$= 116.54$ mm

$M_z/p = 124.722x\ 10^3/2250.686$

$= 63.05$ mm

Both are more than 20mm

2. Assume percentage of steel

(Assume steel larger than required by P and M_x)

$d'/D = 60/600 = 0.1$

$M_x/f_{ck}bd^2 = 262.299\times 10^6 / 30 \times 400 \times 600^2$

$= 0.06$

$P_u/f_{ck}bD = 2250.686 \times 10^3 /30\times 400 \times 600$

$= 0.31$

$p/f_{ck} = 0.018$

Assume a higher value of $p/f_{ck} = 0.02$

$P = 0.020 \times 30 = 0.6$ %

$A_s = 0.6 \times 400 \times 600 / 100$

$= 1440 mm^2$

Use of 6– 18mm Φ bars; As $= 1526.04\ mm^2$

3. Find moment capacities M_x and M_z

About x-axis,

$d'/D = 0.1$, $P/f_{ck}\,bD = 0.31$

$P/fck = 0.02$

$M_{x1}/f_{ck}bd^2 = 0.07$

$M_{x1} = 0.07 \times 30 \times 400 \times 600^2$

$= 302.4$ kNm

About z-axis,

$d'/D = 0.15$, $P/f_{ck}\,bD = 0.31$

$P/fck = 0.02$

$M_{z1}/f_{ck}bd^2 = 0.06$

$M_{z1} = 0.06 \times 30 \times 600 \times 400^2$

$= 172.8$ kN-m

4. Calculate α_n

$P_z = 0.45\, f_{ck}A_c + 0.75\, A_sf_y$

$= 0.45 \times 30 \times 400 \times 600 + 0.75 \times 415 \times 1440$

$= 3688.2$ kN

$P/P_z = 2250.686/ 3688.2$

$= 0.61$

$\alpha_n = 2/3\ [\ 1 + 5/2 \times 0.61]$

$= 1.68$

5. Criterion for bi-axial bending

$(M_x/M_{x1})^{\alpha n} + (M_z/M_{z1})^{\alpha n} <= 1.0$

$(262.299/302.4)^{168} + (124.722/172.8)^{1.68}$

3. Find moment capacities M_x and M_z

About x-axis,

$d'/D = 0.15$, $P/f_{ck}\, bD = 0.39$

$P/fck = 0.15$

$M_{x1}/f_{ck}bd^2 = 0.15$

$M_{x1} = 0.15 \times 25 \times 400 \times 400^2$

$= 240$ kN-m

About z-axis,

$d'/D = 0.15$, $P/f_{ck}\, bD = 0.39$

$P/fck = 0.15$

$M_{z1}/f_{ck}bd^2 = 0.15$

$M_{z1} = 0.15 \times 25 \times 400 \times 400^2$

$= 240$ kN-m

4. Calculate α_n

$P_z = 0.45\, f_{ck}A_c + 0.75\, A_s f_y$

$= 0.45 \times 25 \times 400 \times 400 + 0.75 \times 415 \times 6000$

$= 3667.5$ kN

$P/P_z = 1556.19/ 3667.5$

$= 0.42$

$\alpha_n = 2/3\ [\ 1 + 5/2 \times 0.42]$

$= 1.37$

5. Criterion for bi-axial bending

$(M_x/M_{x1})^{\alpha n} + (M_z/M_{z1})^{\alpha n} <= 1.0$

$(201.229/240)^{1.37} + (98.125/240)^{1.37}$

$= 0.79 + 0.290$

$= 0.99$

<u>Which is less than 1. Hence it's safe</u>

5.3 DESIGN OF SLABS

Design of slab:-

ROOFS

Slab 1 – 3.50×3.52

EFFECTIVE LENGTH –

$l/d = 32$ (continuous slab); $D = 1.50$ m ; $l_x = 3.50 - (0.2 + 0.3)$

$= 3$ m

L_x + depth = 3.150 m

DESIGN LOAD –

Self wt. = 25×150

$= 3.75\ kN/m^2$

$D.L = 1.5\ kN/m^2$ $L.L = 1.5\ kN/m^2$

Total = $1.5 + 1.5 + 3.75$

$= 6.75\ kN/m^2$

Factored load = 1.5×6.75

$= 10.12\ kN/m^2$

MAXIMUM FACTOR MOMENT & CHECKING FOR DEPTH –

$l_y / l_x = 3.52 / 3.5$

$= 1.01$

Table 26 - IS 456

$A_x = 0.048$; $A_y = 0.047$

$M_x = A_x w l_x^2 = 0.048 \times 10.125 \times 3.150^2 = 4.822$ kNm

$M_y = A_y w l_x^2 = 4.721$ kNm

Check depth for max. BM

$M_{max} = 0.138\ f_{ck}\ bd^2$

$d^2 = 4.822 \times 10^6 / 0.138 \times 25 \times 1000$

$37.38 < 150$ mm

`d` is enough.

CHECK FOR SHEAR –

$L_y / l_x = 1.01$

B_x in l_x direction ; B_y in l_y direction

$0.44\ A_x$ in l_x direction; $0.40\ A_x$ in l_y direction

Max. design shear = $A_x\ w\ l_x$

$= 0.44 \times 10.125 \times 3.5$

$= 15.59$ Nm

$V/bd = 15.59 \times 10^3 / 1000 \times 149 = 0.104$

SAFE min. shear for M25 concrete is equal to 0.29 & hence slab is safe.

AREA OF STEEL –

For steel in short direction, d = 149 mm

$M/bd^2 = 4.82 \times 10^6 / 1000 \times 149 \times 149 = 0.217$

Percentage steel, $A_s = 0.084 \times 1000 \times 149/100 = 125.16\ mm^2$

Use 4 bars of 10mm dia.

$A_s = 314\ mm^2$; Spacing = $1000 \times 78.5 / 314 = 250$ mm

$149 - 10 = 139$

$M/bd^2 = 4.721 / 1000 \times 139 \times 139 = 0.244$; $P_t = 0.084$ %

$A_{st} = 0.084 \times 1000 \times 139 / 100 = 116.76\ mm^2$

Use 4 bars of 10 mm dia.

$A_s = 314\ mm^2/m$

CHECK DEFLECTION –

Basic span depth ratio = 26 ; Percent steel along $l_x = 0.084$ %

$F_1 = 2$ (IS 456, fig.4) ; Allowable L/d ratio = $26 \times 2 = 52$

Actual span/depth ratio = $3.5 / 0.149 = 23.49$

Therefore, the assume span depth ratio is enough to control deflection.

Slab 2 – 2.75 × 3.52

EFFECTIVE LENGTH –

l/d = 32 (continuous slab) ; D = 1.50 m

$l_x = 2.75 - (0.2+0.3) = 2.25$ m

L_x + depth = 2.4 m

DESIGN LOAD –

Self wt. = $25 \times 150 = 3.75\ kN/m^2$

D.L = $1.5\ kN/m^2$; L.L = $1.5\ kN/m^2$

Total = $1.5 + 1.5 + 3.75 = 6.75\ kN/m^2$

Factored load = $1.5 \times 6.75 = 10.125\ kN/m^2$

MAXIMUM FACTOR MOMENT & CHECKING FOR DEPTH –

$l_y / l_x = 3.52 / 2.75 \quad = 1.28$

Table 26 - IS 456; $A_x = 0.049$; $A_y = 0.037$

$M_x = A_x w l_x^2 = 0.049 \times 10.125 \times 2.40^2 = 2.857$ kNm

$M_y = A_y w l_x^2 = 2.15$ kNm

Check depth for max. BM

$M_{max} = 0.138\ f_{ck}\ bd^2$

$d^2 = 2.857 \times 10^6 / 0.138 \times 25 \times 1000 = 828.115$

$d = 28.77$; $28.77 < 150$ mm

`d` is enough.

CHECK FOR SHEAR –

$l_y / l_x = 1.28$

B_x in l_x direction ; B_y in l_y direction

0.43 A_x in l_x direction; 0.36 A_x in l_y direction

Max.design shear = $A_x w l_x = 0.43 \times 10.125 \times 2.75 = 11.97$ Nm

$V / bd = 11.97 \times 10^3 / 1000 \times 149 = 0.080$

SAFE min. shear for M25 concrete is equal to 0.29 & hence slab is safe.

AREA OF STEEL –

For steel in short direction, d = 149 mm

$M/bd^2 = 2.857 \times 10^6 / 1000 \times 149 \times 149 = 0.12$

Percentage steel, $A_s = 0.084 \times 1000 \times 149 / 100 = 125.16\ mm^2$

Use 4 bars of 10mm dia.

$A_s = 314\ mm^2$; Spacing = $1000 \times 78.5 / 314 = 250$ mm

$149 - 10 = 139$; $M/bd^2 = 2.15 \times 10^6 / 1000 \times 139 \times 139 = 0.112$

$p_t = 0.084$ %; $A_{st} = 0.084 \times 1000 \times 139 / 100 = 116.76\ mm^2$

Use 4 bars of 10 mm dia.

$A_s = 314\ mm^2/m$

CHECK DEFLECTION –

Basic span depth ratio = 26; Percent steel along $l_x = 0.084$ %

$F_1 = 2$ (IS 456, fig.4); Allowable L/d ratio = $26 \times 2 = 52$

Actual span/depth ratio $= 2.75 / 0.149 = 18.45$

Therefore, the assume span depth ratio is enough to control deflection.

Slab 3 – 3.5 × 4.53

EFFECTIVE LENGTH –

l/d = 32 (continuous slab); D = 1.50 m

$l_x =$ 3.5 - (0.2+0.3) = 3 m

L_x + depth = 3 + 0.15 m = 3.15 m

DESIGN LOAD –

Self wt. = 25×150 = 3.75 kN/m^2

D.L = 1.5 kN/m^2; L.L = 1.5 kN/m^2 ; Total = 1.5 + 1.5 + 3.75

= 6.75 kN/m^2

Factored load = 1.5 × 6.75 = 10.125 kN/m^2

MAXIMUM FACTOR MOMENT & CHECKING FOR DEPTH –

$l_y / l_x =$ 4.53 / 3.5 = 1.29

Table 26 - IS 456; A_x = 0.064; A_y = 0.047

$M_x = A_x w L_x^2$ = 0.064 × 10.125 × 3.15^2 = 6.42 kNm

$M_y = A_y w l_x^2$

= 4.72 kNm

Check depth for max. B.M.

$M_{max} = 0.138\ f_{ck}\ bd^2$

d^2 = 6.42 × 10^6 / 0.138 × 25 × 1000 = 1860.86

d = 43.13 mm; 43.13 < 150 mm

'd' is enough.

CHECK FOR SHEAR –

l_y / l_x = 1.29

B_x in l_x direction ; B_y in l_y direction;

0.48 A_x in l_x direction; 0.40 A_x in l_y direction

Max. design shear = A_x w l_x = 0.48 × 10.125 × 3.5 = 17.01 Nm

V / bd = 17.01 × 10^3 / 1000 × 149 = 0.11

SAFE min. shear for M25 concrete is equal to 0.29 & hence slab is safe.

AREA OF STEEL –

For steel in short direction, d = 149 mm

M/bd^2 = 6.42 × 10^6 / 1000 × 149 × 149 = 0.28

p_t = 0.084 %

Percentage steel, A_s = 0.084 × 1000 × 149/ 100 = 125.16 mm^2

Use 4 bars of 10mm dia.

$A_s = 314\ mm^2$; Spacing = $1000 \times 78.5 / 314 = 250$ mm

$149 - 10 = 139$; $M/bd^2 = 4.72 / 1000 \times 139 \times 139 = 0.24$

$P_t = 0.084$ %

$A_{st} = 0.084 \times 1000 \times 139 / 100 = 116.76\ mm^2$

Use 4 bars of 10 mm dia.

$A_s = 314$ mm^2/m

CHECK DEFLECTION –

Basic span depth ratio = 26; Percent steel along $l_x = 0.084$ %

$F_1 = 2$ (IS 456, fig.4);Allowable L/d ratio = $26 \times 2 = 52$

Actual span/depth ratio $= 3.5 / 0.149 = 23.49$

Therefore, the assume span depth ratio is enough to control deflection.

Slab 4 – 3.33 × 3.87

EFFECTIVE LENGTH –

l/d = 32 (continuous slab); D = 1.50 m

$l_x = 3.33 - (0.2+0.3) = 2.83$ m

L_x + depth = $2.83 + 0.15$ m $= 2.98$ m

DESIGN LOAD –

Self wt. = $25\times150 = 3.75\ kN/m^2$; D.L = 1.5 kN/m ;

L.L. = $1.5\ kN/m^2$

Total = $1.5 + 1.5 + 3.75 = 6.75\ kN/m^2$

Factored load = $1.5 \times 6.75 = 10.125\ kN/m^2$

MAXIMUM FACTOR MOMENT & CHECKING FOR DEPTH –

$l_y / l_x = 3.87 / 3.33 = 1.16$

Table 26 - IS 456; $A_x = 0.048$;$A_y = 0.037$

$M_x = A_x w L_x^2 = 0.048 \times 10.125 \times 2.98^2 = 4.31$ kNm

$M_y = A_y w l_x^2 = 3.326$ kNm

Check depth for max. BM

$M_{max} = 0.138\ f_{ck}\ bd^2$

$d^2 = 4.31 \times 10^6 / 0.138 \times 25 \times 1000 = 1249.27$

d $= 35.34$ mm $= 35.34 < 150$ mm

'd' is enough.

CHECK FOR SHEAR –

$L_y / l_x = 1.16$

B_x in l_x direction ;B_y in l_y direction

0.42 A_x in l_x direction;0.36 A_x in l_y direction

Max. design shear = $A_x w l_x$ = 0.42 × 10.125 × 3.33 = 14.16 Nm

V / bd = 14.16 × 10^3/ 1000 × 149 = 0.095

SAFE min. shear for M25 concrete is equal to 0.29 & hence slab is safe.

AREA OF STEEL –

For steel in short direction, d = 149 mm

M/bd^2 = 4.31 × 10^6 / 1000 × 149 × 149 = 0.19

p_t = 0.084 %;Percentage steel, A_s = 0.084 × 1000 × 149/100

= 125.16mm^2

Use 4 bars of 10mm dia.

A_s = 314 mm^2; Spacing = 1000 × 78.5 / 314 = 250 mm

149 – 10 = 139

M / bd^2 = 3.326 / 1000 × 139 × 139 = 0.172

P_t = 0.084 %; A_{st} = 0.084 × 1000 × 139 / 100 = 116.76 mm^2

Use 4 bars of 10 mm dia.; A_s = 314 mm^2/m

CHECK DEFLECTION –

Basic span depth ratio = 26; Percent steel along l_x = 0.084 %

F_1 = 2 (IS 456, fig.4); Allowable L/d ratio = 26 × 2 = 52

Actual span/depth ratio = 3.33 / 0.149 = 22.34

Therefore, the assume span depth ratio is enough to control deflection.

Slab 5 – 2.10 × 3.52 m

EFFECTIVE LENGTH –

l/d = 32 (continuous slab)

D = 1.50 m; l_x = 2.10 - (0.2+0.3) = 1.6 m

L_x + depth = 1.6 + 0.15 m = 1.75 m

DESIGN LOAD –

Self wt. = 25×150 = 3.75 kN/m^2

D.L = 1.5 kN/m^2; L.L = 1.5 kN/m^2; Total = 1.5 + 1.5 + 3.75

= 6.75 kN/m^2

Factored load = 1.5 × 6.75 = 10.125 kN/m^2

MAXIMUM FACTOR MOMENT & CHECKING FOR DEPTH –

$l_y / l_x = 3.52 / 2.10 = 1.67$

Table 26 - IS 456; $A_x = 0.058$; $A_y = 0.037$

$M_x = A_x w L_x^2 = 0.058 \times 10.125 \times 1.75^2 = 1.798$ kNm

$M_y = A_y w l_x^2 = 1.147$ kNm

Check depth for max. BM

$M_{max} = 0.138\ f_{ck}\ bd^2$

$d^2 = 1.798 \times 10^6 / 0.138 \times 25 \times 1000 = 521.15$

d = 22.82 mm; 22.82 < 150 mm

`d` is enough.

CHECK FOR SHEAR –

$L_y / l_x = 1.67$

B_x in l_x direction ; B_y in l_y direction

0.48 A_x in l_x direction; 0.36 A_x in l_y direction

Max. design shear = $A_x w l_x$ = 0.48 ×10.125×2.10 = 10.206 Nm

V / bd = 10.206 × 10^3/ 1000 × 149 = 0.068

SAFE min. shear for M25 concrete is equal to 0.29 & hence slab is safe.

AREA OF STEEL –

For steel in short direction, d = 149 mm

$M/bd^2 = 1.798 \times 10^6 / 1000 \times 149 \times 149 = 0.080$

p_t = 0.084 %;Percentage steel, A_s = 0.084×1000×149/100

= 125.16 mm^2

Use 4 bars of 10mm dia.

A_s = 314 mm^2; Spacing = 1000 × 78.5 / 314 = 250 mm

149 – 10 = 139; M / bd^2 = 1.147 / 1000 × 139 × 139 = 0.059

P_t = 0.084 %; A_{st} = 0.084 × 1000 × 139 / 100 = 116.76 mm^2

Use 4 bars of 10 mm dia.

A_s = 314 mm^2/m

CHECK DEFLECTION –

Basic span depth ratio = 26; Percent steel along l_x = 0.084 %

$F_1 = 2$ (IS 456, fig.4); Allowable L/d ratio = $26 \times 2 = 52$

Actual span/depth ratio = 2.10 / 0.149 = 14.09

Therefore, the assume span depth ratio is enough to control deflection.

FLOORS

Slab 1 – 3.5 × 3.52

EFFECTIVE LENGTH –

l/d =32 (continuous slab); D = 1.50 m; l_x = 3.5–(0.2+0.3) = 3 m

L_x + depth = 3 + 0.15 m = 3.15 m

DESIGN LOAD –

Self wt. =25×150 =3.75 kN/m^2; D.L = 0.5 kN/m^2; L.L. = 3.5 kN/m^2

Total = 3.5 + 0.5 + 3.75 = 7.75 kN/m^2

Factored load = 1.5 × 7.75 = 11.625 kN/m^2

MAXIMUM FACTOR MOMENT & CHECKING FOR DEPTH –

l_y / l_x = 3.52 / 3.5 = 1.005

Table 26 - IS 456; $A_x = 0.047$; $A_y = 0.047$

$M_x = A_x w L_x^2 = 0.047 \times 10.125 \times 3.15^2 = 5.42$ kNm

$M_y = A_y w l_x^2 = 5.42$ kNm

Check depth for max. BM

$M_{max} = 0.138\, f_{ck}\, bd^2$

$d^2 = 5.42 \times 10^6 / 0.138 \times 25 \times 1000 = 1571.42$

d = 39.64 mm = 39.64 < 150 mm

\`d\` is enough.

CHECK FOR SHEAR –

$l_y / l_x = 1.005$

B_x in l_x direction;B_y in l_y direction

0.40 A_x in l_x direction;0.40 A_x in l_y direction

Max. design shear = $A_x\, w\, l_x$ = 0.40 × 11.625 × 3.5 = 16.275 Nm

V/ bd = 16.275 × 10^3/ 1000 × 149 = 0.109

SAFE min. shear for M25 concrete is equal to 0.29 & hence slab is safe.

AREA OF STEEL –

For steel in short direction, d = 149 mm

$M/bd^2 = 5.42 \times 10^6 / 1000 \times 149 \times 149 = 0.24$

$p_t = 0.084$ %; Percentage steel, $A_s = 0.084 \times 1000 \times 149/100$

$= 125.16\ mm^2$

Use 4 bars of 10mm dia.

$A_s = 314\ mm^2$

Spacing = $1000 \times 78.5 / 314 = 250$ mm

$149 - 10 = 139$; $M/bd^2 = 5.42 / 1000 \times 139 \times 139 = 0.28$

$P_t = 0.084$ %; $A_{st} = 0.084 \times 1000 \times 139 / 100 = 116.76\ mm^2$

Use 4 bars of 10 mm dia.; $A_s = 314\ mm^2/m$

CHECK DEFLECTION –

Basic span depth ratio = 26; Percent steel along $l_x = 0.084$ %

$F_1 = 2$ (IS 456, fig.4); Allowable L/d ratio = $26 \times 2 = 52$

Actual span/depth ratio = $3.5 / 0.149 = 23.49$

Therefore, the assume span depth ratio is enough to control deflection.

Slab 2 – **2.75 × 3.52**

EFFECTIVE LENGTH –

l/d = 32 (continuous slab); D = 1.50 m

$l_x = 2.75 - (0.2+0.3) = 2.25$ m

L_x + depth = 2.25 + 0.15 m = 2.40 m

DESIGN LOAD –

Self wt. = $25 \times 150 = 3.75\ kN/m^2$

D.L = $0.5\ kN/m^2$; L.L = $3.5\ kN/m^2$

Total = $3.5 + 0.5 + 3.75 = 7.75\ kN/m^2$

Factored load = $1.5 \times 7.75 = 11.625\ kN/m^2$

MAXIMUM FACTOR MOMENT & CHECKING FOR DEPTH –

$l_y / l_x = 3.52 / 2.75 = 1.28$

Table 26 - IS 456; $A_x = 0.050$; $A_y = 0.037$

$M_x = A_x w L_x^2 = 0.050 \times 10.125 \times 2.40^2 = 3.348$ kNm

$M_y = A_y w l_x^2 = 2.477$ kNm

Check depth for max. BM

$M_{max} = 0.138\ f_{ck}\ bd^2$

$d^2 = 3.348 \times 10^6 / 0.138 \times 25 \times 1000 = 970.43$

d = 31.15 mm = 31.15 mm < 150 mm

`d` is enough.

CHECK FOR SHEAR –

$L_y / l_x = 1.28; B_x$ in l_x direction ;B_y in l_y direction

0.40 A_x in l_x direction;0.36 A_x in l_y direction

Max. design shear = $A_x\ w\ l_x = 0.40 \times 11.625 \times 2.75 = 12.75$ Nm

$V / bd = 12.75 \times 10^3 / 1000 \times 149 = 0.085$

SAFE min. shear for M25 concrete is equal to 0.29 & hence slab is safe.

AREA OF STEEL –

For steel in short direction, d = 149 mm

$M/bd^2 = 3.348 \times 10^6 / 1000 \times 149 \times 149 = 0.150$

$p_t = 0.084$ %; Percentage steel, $A_s = 0.084 \times 1000 \times 149/100$

$= 125.16\ mm^2$

Use 4 bars of 10mm dia.

$A_s = 314\ mm^2$; Spacing = $1000 \times 78.5 / 314 = 250$ mm

$149 - 10 = 139$; $M/ bd^2 = 2.477 / 1000 \times 139 \times 139 = 0.12$

$P_t = 0.084$ %; $A_{st} = 0.084 \times 1000 \times 139 / 100 = 116.76\ mm^2$

Use 4 bars of 10 mm dia.

$A_s = 314\ mm^2/m$

CHECK DEFLECTION –

Basic span depth ratio = 26; Percent steel along $l_x = 0.084$ %

$F_1 = 2$ (IS 456, fig.4); Allowable L/d ratio = $26 \times 2 = 52$

Actual span/depth ratio = $2.75 / 0.149 = 18.45$

Therefore, the assume span depth ratio is enough to control deflection.

Slab 3 – 3.5 ×4.53

EFFECTIVE LENGTH –

l/d = 32 (continuous slab);D = 1.50 m; $l_x = 3.5 - (0.2+0.3) = 3$ m

L_x + depth = 3 + 0.15 m = 3.15 m

DESIGN LOAD –

Self wt. = 25×150 = 3.75 kN/m^2

D.L = 0.5 kN/m^2;L.L = 3.5 kN/m^2;Total = 3.5 + 0.5 + 3.75

= 7.75 kN/m^2

Factored load = 1.5 × 7.75 = 11.625 kN/m^2

MAXIMUM FACTOR MOMENT & CHECKING FOR DEPTH –

$l_{y}/ l_x = 4.53 / 3.5 = 1.29$

Table 26 - IS 456;$A_x = 0.064$;$A_y = 0.047$

$M_x = A_x w L_x^2 = 0.064 \times 10.125 \times 3.15^2 = 7.382$ kNm

$M_y = A_y w l_x^2 = 5.42$ kNm

Check depth for max. BM

$M_{max} = 0.138\ f_{ck}\ bd^2$

$d^2 = 7.382 \times 10^6 / 0.138 \times 25 \times 1000 = 2139.71$

d = 46.25 mm = 46.25mm < 150 mm

'd' is enough.

CHECK FOR SHEAR –

$L_y / l_x = 1.29$

B_x in l_x direction ;B_y in l_y direction

0.49 A_x in l_x direction;0.40 A_x in l_y direction

Max. design shear = $A_x\ w\ l_x = 0.49 \times 11.625 \times 3.5 = 19.93$ Nm

$V/ bd = 19.93 \times 10^3 / 1000 \times 149 = 0.13$

SAFE min. shear for M25 concrete is equal to 0.29 & hence slab is safe.

AREA OF STEEL –

For steel in short direction, d = 149 mm

$M/bd^2 = 7.382 \times 10^6 / 1000 \times 149 \times 149 = 0.33$

$p_t = 0.090$ % ;Percentage steel, $A_s = 0.090 \times 1000 \times 149/100$

= 134.16 mm^2

Use 4 bars of 10mm dia.

$A_s = 314$ mm^2; Spacing = 1000 × 78.5 / 314 = 250 mm

149 – 10 = 139; $M/ bd^2 = 5.42 / 1000 \times 139 \times 139 = 0.28$

$P_t = 0.084$ %;$A_{st} = 0.084 \times 1000 \times 139 / 100 = 116.76$ mm^2

Use 4 bars of 10 mm dia.

$A_s = 314$ mm^2/m

CHECK DEFLECTION –

Basic span depth ratio = 26;Percent steel along $l_x = 0.084$ %

$F_1 = 2$ (IS 456, fig.4);Allowable L/d ratio = $26 \times 2 = 52$

Actual span/depth ratio $= 3.5 / 0.149 = 23.49$

Therefore, the assume span depth ratio is enough to control deflection.

Slab 4 – 3.33 × 3.8

EFFECTIVE LENGTH –

l/d = 32 (continuous slab);D = 1.50 m

$l_x = 3.33 - (0.2+0.3) = 2.83$ m

l_x + depth = 2.83 + 0.15 m = 2.98 m

DESIGN LOAD –

Self wt. = 25×150 = 3.75 kN/m^2; DL = 0.5 kN/m^2; LL = 3.5 kN/m^2

Total = 3.5 + 0.5 + 3.75 = 7.75 kN/m^2;

Factored load = 1.5 × 7.75 = 11.625 kN/m^2

MAXIMUM FACTOR MOMENT & CHECKING FOR DEPTH –

$l_y / l_x = 3.80 / 3.33 = 1.14$

Table 26 - IS 456;$A_x = 0.046$;$A_y = 0.037$

$M_x = A_x w L_x^2 = 0.046 \times 10.125 \times 2.98^2 = 4.74$ kNm

$M_y = A_y w l_x^2 = 3.819$ kNm

Check depth for max. BM

$M_{max} = 0.138 f_{ck} bd^2$

$d^2 = 4.74 \times 10^6 / 0.138 \times 25 \times 1000 = 1373.91$

d = 37.06 mm = 37.06mm < 150 mm

\`d\` is enough.

CHECK FOR SHEAR –

$L_y / l_x = 1.14$

B_x in l_x direction ;B_y in l_y direction

0.42 A_x in l_x direction;0.36 A_x in l_y direction

Max. design shear = A_x w l_x = 0.42 × 11.625 × 3.33 = 16.25 Nm

V/ bd = $16.25 \times 10^3 / 1000 \times 149 = 0.109$

SAFE min. shear for M25 concrete is equal to 0.29 and hence slab is safe.

AREA OF STEEL –

For steel in short direction, d = 149 mm

$M/bd^2 = 4.74 \times 10^6 / 1000 \times 149 \times 149 = 0.213$

$p_t = 0.084$ %;Percentage steel, $A_s = 0.084 \times 1000 \times 149/100$

$= 125.16\ mm^2$

∴ Use 4 bars of 10mm dia.

$A_s = 314\ mm^2$;Spacing = $1000 \times 78.5 / 314 = 250$ mm

$149 - 10 = 139$;$M/bd^2 = 3.819 / 1000 \times 139 \times 139 = 0.197$

$P_t = 0.084$ %; $A_{st} = 0.084 \times 1000 \times 139 / 100 = 116.76\ mm^2$

Use 4 bars of 10 mm dia.

$A_s = 314\ mm^2/m$

CHECK DEFLECTION –

Basic span depth ratio = 26;Percent steel along $l_x = 0.084$ %

$F_1 = 2$ (IS 456, fig.4);Allowable L/d ratio = $26 \times 2 = 52$

Actual span/depth ratio = $3.33 / 0.149 = 22.34$

Therefore, the assume span depth ratio is enough to control deflection.

Slab 5 – 2.10 × 3.52

EFFECTIVE LENGTH –

l/d = 32 (continuous slab);D = 1.50 m

$l_x = 2.10 - (0.2+0.3) = 1.6$ m

l_x + depth = 1.6 + 0.15 m = 1.75 m

DESIGN LOAD –

Self wt. = 25×150 = 3.75 kN/m^2 ;D.L = 0.5 kN/m^2 ; L.L.

= 3.5 kN/m^2

Total = 3.5 + 0.5 + 3.75 = 7.75 kN/m^2;

Factored load = 1.5 × 7.75

= 11.625 kN/m^2

MAXIMUM FACTOR MOMENT & CHECKING FOR DEPTH –

$l_y / l_x = 3.52 / 2.10 = 1.676$

Table 26 - IS 456; $A_x = 0.060$;$A_y = 0.037$

$M_x = A_x w L_x^2 = 0.060 \times 11.625 \times 1.75^2 = 1.22$ kNm

$M_y = A_y w l_x^2 = 0.752$ kNm

Check depth for max. BM

$M_{max} = 0.138\ f_{ck}\ bd^2$

$d^2 = 1.22 \times 10^6 / 0.138 \times 25 \times 1000 = 353.62$

$d = 18.80$ mm = 18.80mm < 150 mm

'd' is enough.

CHECK FOR SHEAR –

$L_y / l_x = 1.676$; Bx in l_x direction ;B_y in l_y direction

0.48 A_x in l_x direction; 0.36 A_x in l_y direction

Max. Design shear = $A_x\ w\ l_x = 0.48 \times 11.625 \times 2.10$

= 24.607 Nm

$V/bd = 24.607 \times 10^3 / 1000 \times 149 = 0.165$

SAFE min. shear for M25 concrete is equal to 0.29 & hence slab is safe.

AREA OF STEEL –

For steel in short direction, d = 149 mm

$M/bd^2 = 1.22 \times 10^6 / 1000 \times 149 \times 149 = 0.054$

$p_t = 0.084$ %;Percentage steel, A_s = $0.084 \times 1000 \times 149/100$

= 125.16 mm^2

Use 4 bars of 10mm dia.

$A_s = 314$ mm^2; Spacing = $1000 \times 78.5 / 314 = 250$ mm

$149 - 10 = 139$; $M/bd^2 = 0.752 / 1000 \times 139 \times 139 = 0.038$

$p_t = 0.084$ %;$A_{st} = 0.084 \times 1000 \times 139 / 100 = 116.76$ mm^2

Use 4 bars of 10 mm dia.;$A_s = 314$ mm^2/m

CHECK DEFLECTION –

Basic span depth ratio = 26; Percent steel along, $l_x = 0.084$ %

$F_1 = 2$ (IS 456, fig.4); Allowable L/d ratio = $26 \times 2 = 52$

Actual span/depth ratio = 2.10 / 0.149 = 14.09

Therefore, the assume span depth ratio is enough to control deflection.

5.4 COMPARISON OF MANUAL AND STAAD.PRO DESIGN OUTPUT

Table 5.1 shows the areas of steel given by STAAD.pro and those obtained by manual design using SP-16 (IS-456 : 2000).

Table 5.1

Member	Area of steel from STAAD.Pro (mm^2)	Area of steel obtained manually (mm^2)	Percentage Difference
Column group 1	3727	3780	1.40%
Column group 2	1419	1440	1.15%
Column group 3	5883	6000	1.95%
Beam group 1	1611	1621	0.66%
Beam group 2	1833	1867	1.82%
Beam group 3	1171	1186	1.26%
Beam group 4	800	1929	58.5%*
Beam group 5	594	610	2.62%

All the values from STAAD.Pro are close to the values calculated manually hence the results are verified.

* The error % is too high in this case. This may be attributed to the inability of STAAD.pro to take up design of very short beams (length here being 1m approx.) with good accuracy.

Chapter 6

Reinforcement Detailing and Drawing

6.1 GENERAL

The reinforcement detailing has been done as per IS 13920- 1993 so that ductility in the building can be achieved. The detailed reinforcement drawings are attached with this report.

6.1.1 TYPICAL DUCTILE DETAILING OF VARIOUS ELEMENTS

For sake of reference typical details of beam, column, and beam column joint are shown along with.

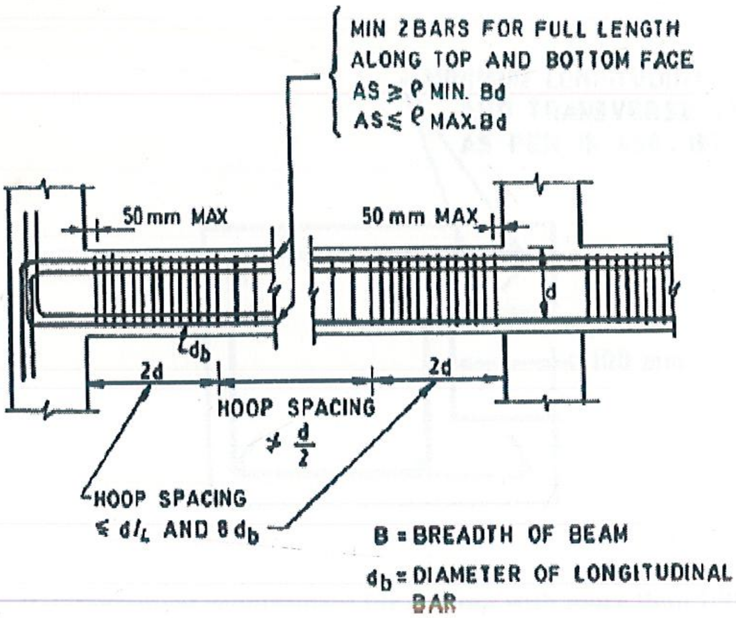


Fig 6.1: Beam Reinforcement as per IS-13920:1993

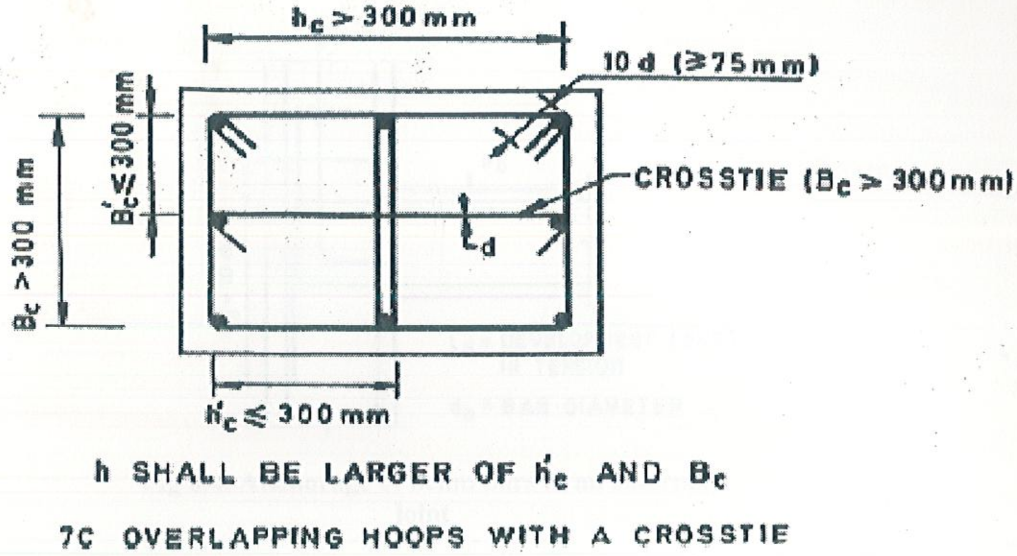


Fig 6.2: Transverse Reinforcement in Column

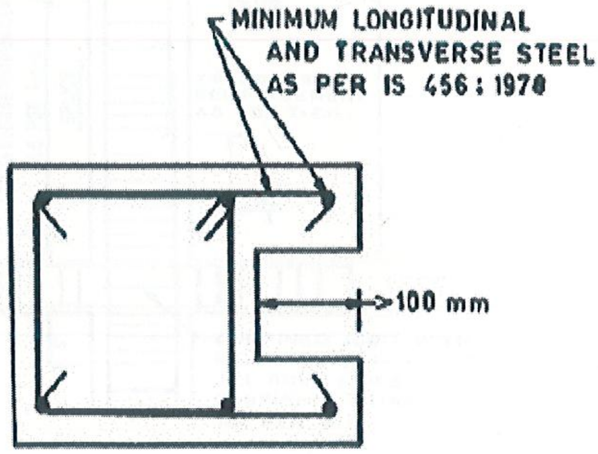


Fig 6.3: Reinforcement requirement for column with more than 100mm projection beyond core.

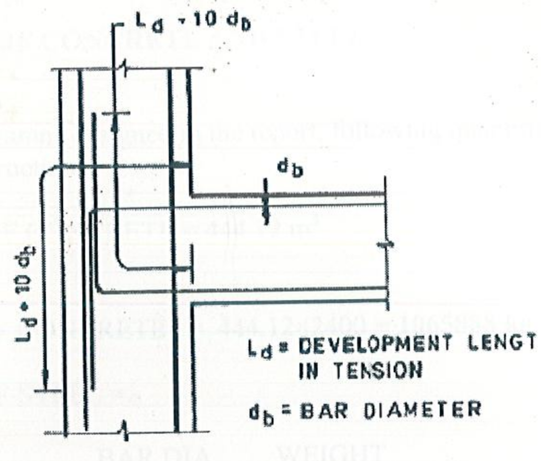


Fig 6.4: Anchorage of beam bars in an external joint

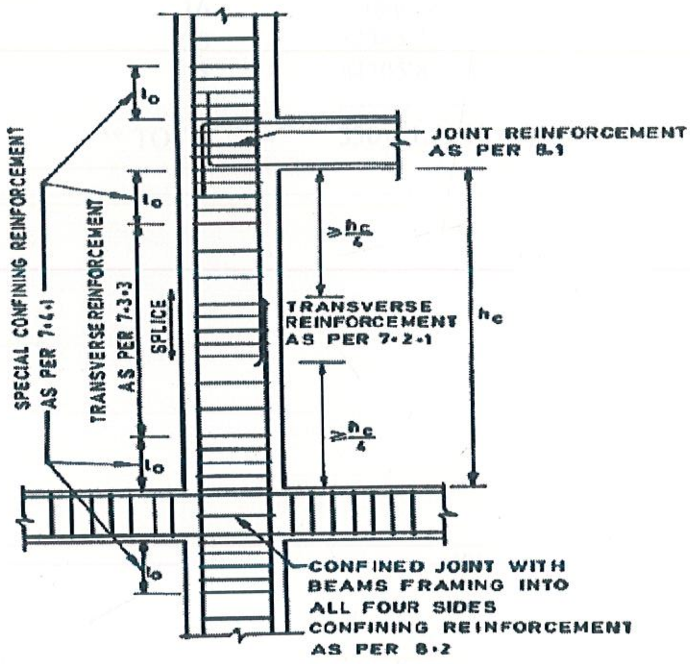


Fig 6.5: Typical Column and joint detailing

6.2 QUANTITIES OF CONCRETE AND STEEL

For all beams and Columns designed in the report, following quantities of materials are required for construction:

TOTAL VOLUME OF CONCRETE = 444.12 m^3

TOTAL WEIGHT OF CONCRETE = 444.12×2400 = **1065888 kg = 1065 tons**

TOTAL WEIGHT OF STEEL =

BAR DIA (in mm)	WEIGHT (in Newtons)
8	67321.20
10	55522.68
12	111590.48
16	59066.28
20	22385.32
25	14305.88
*** TOTAL =	330191.81 N = approx. **33 tons steel**

CONCLUSIONS

In this project we have analyzed and designed a multi-storey building of 15-storey with the help of STAAD-pro. We have analyzed each and every aspect of the building so that building should not collapse except non-structural damages that will be minor and can be repaired easily. After knowing deflection and moment of the building frame when we applied the earthquake forces and the loads, we came to know the behavior of the building, which is very much satisfactory and reliable.

Analysis of the Multi-storey building was done with the help of STAAD.pro and it was observed after manual analysis of representative beams and columns from every group classified on the basis of length, grade of concrete and loads that the values are in concurrence with the values yielded by the software.

The Analysis and Design confirms that the building is safe and satisfies the safety and serviceability criteria as it has been designed by the limit state method. Also, the amount of concrete and steel needed to construct the building was calculated with the help of STAAD.Pro.

BIBLIOGRAPHY

1. IS-875:1987 PART-1: *Dead Loads- Unit weights of building materials and stored materials*
2. IS-875:1987 PART-2 *Imposed Loads*
3. IS-875:1987 PART-3 *wind load*
4. IS-1893:2002 PART-1 *Criteria for earthquake resistant design of structures(General provisions and buildings)*
5. IS-2911: 1980 PART-3 *Pile foundation*
6. IS-456-2000 *Plain and reinforced concrete*
7. IS-13920-1993 *Ductile detailing of reinforced concrete structures subjected to seismic forces*
8. SP-16: *Design Aids for IS 456: 2000*
9. Limit State Design of reinforced Concrete Structures by PC Varghese
10. Earthquake Resistant Design of Structures by Agarwal and Shrikhande published by Prentice-Hall India
11. Reinforced Concrete (limit state design) fifth edition by Ashok K. Jain published by Nem Chand & Bros, Roorkee
12. Design of RCC Stuctural Elements Volume-1 by S.S. Bhavikatti Published by New Age International.