# ANALYSIS AND DESIGN OF
# STEEL – CONCRETE COMPOSITE STRUCTURE

**A PROJECT REPORT**

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## BONAFIDE CERTIFICATE

Certified that this project report **"ANALYSIS AND DESIGN OF STEEL - CONCRETE COMPOSITE STRUCTURE "** is the bonafide work of **"RAMYAA .B (41501103012), PRASATH .P (41501103010) and VENKADESAN .S (41501103021)"** who carried out the project work under my supervision.

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## ABSTRACT OF THE PROJECT WORK

The project involves Planning, Analysis and Design of an Institutional Building with steel-concrete composite construction. The proposed structure is a **G+5** building, with **3.658m** as the height of each floor. The overall plan dimension of the building is 56.3m x 31.94m.

The analysis and design involves the structural planning, load calculation, analyzing it by 2D modeling using STAAD-Pro 2003, design of composite floors and columns, design of steel beams and design of foundation. Analysis has been done for various load combinations including seismic load, wind load, etc, as per the Indian Standard Code of Practice.

The project also involves analysis and design of an equivalent R.C.C. structure so that a cost comparison can be made between a steel-concrete composite structure and an equivalent R.C.C. structure.

# TABLE OF CONTENTS


| CHAPTER NO | TITLE | PAGE NO |
|---|---|---|
| | **Acknowledgement** | **iii** |
| | **Abstract of the Project work ( Tamil )** | **iv** |
| | **Abstract of the Project work ( English )** | **v** |
| | **List of Tables** | **viii** |
| | **List of Figures** | **ix** |
| **1** | **INTRODUCTION** | **1** |
| | 1.1 PROJECT OBJECTIVE | 1 |
| | 1.2 COMPOSITE STRUCTURES | 1 |
| | 1. 3 NEED OF STEEL IN CONSTRUCTION | 2 |
| | 1.4 NEED OF STEEL -CONCRETE COMPOSITE SECTION | 3 |
| | 1.5 ADVANTAGES OF COMPOSITE CONSTRUCTION | 3 |
| **2** | **COMPOSITE MULTISTOREYED BUILDING** | **5** |
| | 2.1 ELEMENTS OF COMPOSITE MULTISTOREYED BUILDING | 5 |
| | 2.1.1 SHEAR CONNECTORS | 5 |
| | 2.1.2 PROFILED DECK | 9 |
| | 2.1.3 COMPOSITE BEAMS | 19 |
| | 2.1.4 ENCASED COLUMNS | 21 |

| | | |
|---|---|---|
| **3** | **LOAD CALCULATIONS** | **27** |
| | 3.1 DEAD LOAD | 27 |
| | 3.2 LIVE LOAD | 27 |
| | 3.3 WIND LOAD | 28 |
| | 3.4 SEISMIC LOAD | 30 |
| **4** | **ANALYSIS** | **32** |
| **5** | **DESIGN OF ELEMENTS ( COMPOSITE )** | **42** |
| | 5.1 DESIGN OF SLABS | 42 |
| | 5.2 DESIGN OF BEAMS | 46 |
| | 5.3 DESIGN OF COLUMNS | 54 |
| | 5.4 DESIGN OF FOUNDATION | 65 |
| **6** | **DESIGN OF ELEMENTS ( R.C.C. )** | **68** |
| | 6.1 DESIGN OF BEAMS | 68 |
| | 6.2 DESIGN OF COLUMNS | 70 |
| | 6.3 DESIGN OF FORM WORK | 70 |
| **7** | **COST COMPARISON** | 71 |
| 8 | **CONCLUSION** | **73** |
| | **REFERENCES** | **74** |

# LIST OF TABLES


| TABLE NO | TITLE | PAGE NO |
|---|---|---|
| 3.3.a | WIND LOADS-PARALLEL TO SHORTER DIRECTION | 29 |
| 3.3.b | WIND LOADS-PARALLEL TO LONGER DIRECTION | 30 |
| 3.3.c | CALCULATION OF LATERAL FORCES | 31 |
| 7.a | COMPARISON OF COMPOSITE SLABS WITH CONVENTIONAL RCC SLABS | 71 |
| 7.b | COMPARISON OF COMPOSITE BEAMS WITH CONVENTIONAL RCC BEAMS (longer span) | 71 |
| 7.c | COMPARISON OF COMPOSITE BEAMS WITH CONVENTIONAL RCC BEAMS (shorter span) | 72 |
| 7.d | COMPARISON OF COMPOSITE COLUMNS WITH CONVENTIONAL RCC COLUMNS | 72 |

## LIST OF FIGURES


| SL. NO | TITLE | PAGE NO |
|---|---|---|
| 3.3.i | PLAN AND ELEVATION | 28 |
| 4.i | BENDING MOMENT DIAGRAM FOR BEAM | 40 |
| 4.ii | BENDING MOMENT DIAGRAM FOR COLUMN | 41 |
| 5.1.i | CROSS SECTION OF PROFILED DECK SLAB | 42 |

# CHAPTER 1

# INTRODUCTION

## 1.1 PROJECT OBJECTIVE

The use of Steel in construction industry is very low in India compared to many developing countries. Experiences of other countries indicate that this is not due to the lack of economy of Steel as a construction material. There is a great potential for increasing the volume of Steel in construction, especially in the current development needs in India. Not exploring Steel as an alternative construction material and not using it where it is economical is a heavy loss for the country. Also, it is evident that now-a-days, the composite sections using Steel encased with Concrete are economic, cost and time effective solution in major civil structures such as bridges and high rise buildings.

In due consideration of the above fact, this project has been envisaged which consists of analysis and design of a high rise building using Steel-Concrete composites. The project also involves analysis and design of an equivalent R.C.C structure so that a cost comparison can be made between a Steel-Concrete composite structure and an equivalent R.C.C. structure.

## 1.2 COMPOSITE STRUCTURES

Composite Steel-Concrete Structures are used widely in modern bridge and building construction. A composite member is formed when a steel component ,such as an I beam ,is attached to a concrete component, such as a floor slab or bridge deck. In such a composite T-beam the comparatively high strength of the concrete in compression complements the high strength of the steel in tension. The fact that each material is used to the fullest advantage makes composite Steel-Concrete construction very efficient and economical. However, the real attraction of such construction is based on having an efficient connection of the Steel to the Concrete, and it is this connection that allows a transfer of forces and gives composite members their unique behavior.

## 1.3 NEED OF STEEL IN CONSTRUCTION

In building construction, role of steel is same as that of bones in a living being. Steel is very advantageous because it:

- Offers considerable flexibility in design and is easy for fabrication
- Facilities faster construction scheduling of projects
- Enables easy construction scheduling even in congested sites
- Permits large span construction repair/modification
- Is an ideal material in earthquake prone locations due to high strength, stiffness, ductility
- Is environment friendly and fully recyclable on replacement

Abroad, the use of structural steel has been growing, and has now become one of the important input materials of construction. In India, until nineties, availability of structural steel was in less and weather resistant and /or strength grades were not readily available. Thus, steel did not make much in-roads in building construction and highways, and its share in bridge construction also started decreasing. This coupled with many other reasons led to stagnation of steel demand, while large-scale production capacity has been created in the country during initial liberalization period of our country. Hence, proper development of steel application sectors has become an important issue and the steel framed composite construction is considered to be a cost effective solution for multi-storied buildings due to optimum use of materials.

## 1.4 NEED OF STEEL - CONCRETE COMPOSITE SECTION

Steel concrete composite construction combines the compressive strength of concrete with the tensile strength of steel to evolve an effective and economic structural system. Over the years, this specialized field of construction has become more and more popular in the western world and developed into a multifaceted design and construction technique. Apart from composite beam, slab and column, options like composite truss, slim-floor etc are also being explored in the field of composite construction.

## 1.5 ADVANTAGE OF COMPOSITE CONSTRUCTION

In conventional composite construction, concrete slabs rest over steel beams and are supported by them. Under load, these two components act independently and a relative slip occurs at the interface if there is no connection between them. With the help of a deliberate and appropriate connection provided between the beam and the concrete slab, the slip between them can be eliminated. In this case, the steel beam and the slab act as a “composite beam” and their action is similar to that of a monolithic Tee beam. Since concrete is stronger in compression than in tension, and steel is susceptible to buckling in compression, by the composite action between the two, we can utilize their respective advantages to the fullest extent. There are many advantages associated with steel-concrete composite construction. Some of these are listed below:

- The most effective utilization of steel and concrete is achieved.
- Keeping the span and loading unaltered, a more economical steel section (in terms of depth and weight) is achievable in composite construction compared with conventional non-composite construction.
- As the depth of beam reduces, the construction depth reduces, resulting in enhanced headroom.
- Because of its larger stiffness, composite beams have less deflection than steel beams.
- Composite construction is amenable to "fast-track" construction because of using rolled steel and pre-fabricated components, rather than case-in-situ concrete.
- Encased steel beam sections have improved fire resistance and corrosion.
- Considerable flexibility in design, pre-fabrication and construction scheduling in congested areas.

# CHAPTER 2

# COMPOSITE MULTISTOREYED BUILDING

## 2.1 ELEMENTS OF COMPOSITE MULTISTOREYED BUILDING

### 2.1.1 SHEAR CONNECTORS

Shear connections are essential for steel concrete construction as they integrate the compression capacity of supported concrete slab with supporting steel beams / girders to improve the load carrying capacity as well as overall rigidity. Though steel to concrete bond may help shear transfer between the two to certain extent, yet it is neglected as per the codes because of its uncertainty. All codes therefore, specify positive connectors at the interface of steel and concrete.

**Effect of Shear Connection on Bending and Shear Stresses**

The behaviour of composite beams under transverse loading is in two extreme cases where a) a full interaction (b) with no interaction, can best be illustrated by using two identical beams, each having a cross section of b x h and spanning a distance of l, one placed at the top of the other. The beams support a uniformly distributed load of w/unit length. For theoretical explanation, two extreme cases of no interaction and 100%(full) interaction are analyzed

**a. No Interaction Case**

It first assumed that there is no shear connection between the beams, so that they are just seated on one another but act independently. The moment in inertia (I) of each beam is given by $bh^3/12$. The load carried by each beam is w/2 per unit length, with mid span moment of $wl^2/16$ and vertical compressive stress of w/2b at the interface. From elementary beam theory, the maximum bending stress in each beam is given by,

$$f = \frac{M_{y_{max}}}{I} = \frac{3wl^2}{8bh^2}$$

Where, M is the maximum bending moment and $y_{max}$ is the distance to the extreme fibre equal to h/2. The maximum shear stress ($q_{max}$) that occurs at the neutral axis of each member near support is given by

$$q_{max} = \frac{3wl}{8bh}$$

and the maximum deflection is given by

$$\delta = \frac{5wl^4}{64Ebh^3}$$

The bending moment in each beam at a distance x from mid span is, $M_x = w(l^2-4x^2)/16$

So, the tensile strain at the bottom fibre of the upper beam and the compression stress at the top fibre of the lower beam is,

$$\varepsilon_x = \frac{My_{max}}{EI} = \frac{3w(l^2-4x^2)}{8Ebh^2}$$

Hence, the top fibre of the bottom beam undergoes slip relative to the bottom fibre of the top beam. The slip strain i.e. the relative displacement between adjacent fibres is therefore 2ε Denoting slip by S, we get.

$$\frac{dS}{dx} = 2\varepsilon_x = \frac{3w(l^2-4x^2)}{4Ebh^2}$$

Integrating and applying the symmetry boundary conditions S = 0 at x=0 we get the equation

$$S = \frac{w\left(3l^2x - 4x^3\right)}{4Ebh^2}$$

The maximum slip (i.e. $S_{max} = wl^2/4Ebh^2$) works out to be 3.2h/l times the maximum deflection of each beam derived earlier. If l/(2h) of beams is 20, the slip value obtained is 0.08 times the maximum deflection. This shows that slip is very small in comparison to deflection of beam. In order to prevent slip between the two beams at the interface and ensure bending strain compatibility, shear connectors are frequently used. Since the slip at the interface is small these shear connections, for full composite action, have to be very stiff.

### b. Full (100%) interaction case

Let us now assume that the beams are joined together by infinitely stiff shear connection along the face AB in Fig. 1. As slip strain are now zero everywhere, this case is called "full interaction". In this case, the depth of the composite beam is twoh with a breadth b, so that $I=2bh^3/3$. The mid-span moment is $wl^2/8$. The maximum bending stress is given by

$$f_{max} = \frac{My_{max}}{I} = \frac{3wl^2}{16bh^2}$$

This value is half of the bending stress given for "no interaction case". The maximum shear stress $q_{max}$ remains unaltered but occurs at mid depth. The mid span deflection is

$$\delta = \frac{5wl^4}{256Ebh^3}$$

This value of deflection is one fourth of that of the value obtained from no interaction case

Thus, by providing fully shear connection between slab and beam, the strength and stiffness of the system can be significantly increased, even though the material consumption is essentially the same.

**Types of Shear Connectors**

The total shear force at the interface between a concrete slab and steel beam is approximately eight times the total load carried by the beam. Therefore, mechanical shear connectors are required at the steel-concrete interface. These connectors are designed to (a) transmit longitudinal shear along the interface and, (b) prevent separation of steel beam and concrete slab at the interface.

Thus, mechanical shear connectors are provided to transmit the horizontal shear between the steel beam and the concrete slab, ignoring the effect of any bond between the two. It also resists uplift force acting at the steel interface. Commonly used types of shear connectors as per IS: 11384 – 1985: Code of practice for composite construction in structural steel and concrete.

There are three main types of shear connectors; rigid shear connectors, flexible shear connectors and anchorage shear connectors. These are explained below:

<u>Rigid Shear Connectors</u>

As the name implies, these connectors are very stiff and they sustain only a small deformation while resisting the shear force. They derive their resistance from bearing pressure on the concrete, and fail due to crushing of concrete. Short bars, angles T-sections are common examples of this type of connectors. In addition, anchorage devices like hopped bars are attached with these connectors to prevent vertical separation.

Flexible Shear Connectors

Flexible shear connectors consist of headed studs, channels or tees welded to the top flange of the steel beams come under this category. They derive their stress resistance through bending and undergo large deformation before failure. The stud connectors are the types used extensively. The shank and weld collar adjacent to steel resist the shear loads whereas the head resists the uplift.

Anchorage Shear Connectors

Anchorage type shear connector is used to resist longitudinal shear and to prevent separation if the beam/ girder from the concrete slab at the interface through bond. In this case, mild steel inclined rods or steel rods in the form of helical stirrups are welded on the top flange of the steel beam.

### 2.1.2 PROFILED DECK

**Advantages of Profiled deck**

Composite floors using profiled sheet decking have become very popular in the West for high-rise buildings. Composite deck slabs are generally competitive where the concrete floor has to be completed quickly and where medium level of fire protection to steel work is sufficient.

There is presently no Indian standard covering the design of composite floor systems using profiled sheeting.

In composite floors, the structural behavior is similar to a reinforced concrete slab, with the steel sheeting acting as the tension reinforcement. The main structural and other benefits of using composite floors with profiled steel decking are:

- Savings in steel weight are typically 30% to 50% over non-composite construction

- Greater stiffness of composite beams results in shallower depths for the same span. Hence lower stored heights are adequate resulting in savings in classing costs, reduction in wind loading and savings in foundation costs Faster rate of construction

The steel decking performs a number of roles, such as:

- It supports loads during construction and acts as a working platform

- It develops adequate composite action with concrete to resist the imposed loading

- It transfers in-plane loading by diaphragm action to vertical bracing or shear walls

- It stabilizes the volume of concrete in tension zone.

- It distributes shrinkage strains, thus preventing serious cracking of concrete.

Excessive ponding in long span composite floors shall be avoided by providing required propping. Otherwise, the profiled sheet deflects considerably requiring additional concrete at the center that may add to the concreting cost.

**Structural Elements of Profiled Slab**

Composite floors with profiled decking consist of the following structural elements along with in-situ concrete and steel beams:

- Profiled decking
- Stud shear connectors
- Reinforcement for shrinkage and temperature stresses

Stud shear connectors are invariably used in composite floors. Stud shear connectors are welded through the sheeting on to the top flange of the beam. Insulation requirements for fire usually control the slab thickness above the profile. Thickness values between 65 and 120mm are sufficient to give a fire rating of up to 2 hours. Lightweight concrete is generally preferred in composite floors due to reduced weight on profiled sheets and enhanced fire-insulation.

**Profiled Sheet Decking**

The steel deck is normally rolled into the desired profile from 22G (0.70mm) to 16G (1.6mm) galvanized coil. It is profiled such that the profile heights are usually in the range of 40 – 60 mm whereas higher depth of 85mm is also available. The typical trough width lies between 150 to 350mm. Generally, spans of the order of 2.5m to 3.5m between the beams are chosen and the beams are designed to span between 6m to 12m. There are two well-Known generic types of profiles.

- Dovetail profile
- Trapezoidal profile with web indentations

Profiled deck shaped are chosen based on the ability to enhance the bond at the steel-concrete concrete interface and providing stability while supporting wet concrete and other construction loads.

Indentations and protrusions into the rib mobilize the bearing resistance in addition to adhesion and also provide the shear transfer in composite slabs.

**Profiled Sheeting as permanent Formwork**

The role and performance of profiled steel deck at various stages of composite action is explained below. During construction stage, it acts as temporary formwork. Also, it carries the weight of wet concrete, self-weight, workers and equipments. The profile deck must be strong enough to carry this load and stiff enough to be serviceable under the weight of wet concrete.

**Composite beam stage:**

The composite beam formed by employing the profiled steel sheeting is different from the one with a normal solid slab, as the profiling would influence its strength and stiffness. This is termed '*composite beam stage'*. In this case, the profiled deck, which is fixed transverse to the beam, results in voids within the depth of the associated slab. Thus, the area of concrete used in calculating the section properties can only be that depth of slab above the top flange of the profile. In addition, any stud connector welded through the sheeting must lie within the area of concrete in the trough of the profiling. Consequently, if trough is narrow, reduction in strength must be made because of the reduction in area of constraining concrete. In current design methods, conservatively the steel sheeting is ignored while calculating shear resistance.

**Composite slab Stage**

The structural behaviour of the composite slab is similar to that of a reinforced concrete beam with no shear reinforcement. The steel sheeting provides adequate tensile capacity in order to act with no shear reinforcement. The shear between the steel and concrete must be carried by friction and bond between the materials. The mechanical keying action of the embossments is of great importance. This is especially so in open trapezoidal profiles, where the embossments must also provide resistance to vertical separation. In addition to structural adequacy, the finished slab must be capable of satisfying the requirements of fire resistance.

**Design Method**

As there is no Indian standard covering profile decking, Eurocode 4 (EC4) provisions are considered. The design method defined in EC4 requires that slab be checked firstly for bending capacity, assuming full bond between concrete and steel, secondly for shear bond capacity and, finally, for vertical shear. The analysis of the bending capacity of the slab may be carried out as though the slab was of reinforced concrete with the steel deck acting as reinforcement. However, no satisfactory analytical method has been developed as far for estimating the value shear bond capacity. Based on test data available, the loads at the construction stage often govern the allowable span rather at the composite slab stage.

**Shear Connectors**

Shear connectors are steel elements such as studs, bars, spiral or ant other similar devices welded to the top flange of the steel section and intended to transmit the horizontal shear between the steel section and the cast in-situ concrete and also to prevent vertical separation at the interface.

### Reinforcement for Shrinkage and Temperature Stresses

In buildings, temperature difference in the slabs is negligible; thus there is no need to provide reinforcement to account for temperature stresses. The effect of shrinkage is considered and the total shrinkage strain for design may be taken as 0.003 in the absence of the test data.

### Bending Resistance of Composite Slab

The structural properties of profiled sheet along with reinforcement provided and concrete with a positive type of interlock between concrete and steel deck is the basis of a composite floor. Some loss of interaction and hence slip may occur between concrete – steel interface. Such a case is known as *'partial interaction'* Failure in such cases due to a combination of flexure and shear.

The width of the slab 'b' is one typical wavelength of profiled sheeting. The overall thickness is ht and the depth of concrete above main flat surface is hc. Normally , ht is not less than 80 mm and hc is not less than 40 mm from sound and fire insulation considerations.

The neutral axis normally lies in the concrete in case of full shear connection; but in regions of partial shear connection, the neutral axis may be within the steel section, in which case local buckling of steel should be checked.

For sheeting in tension, the width of embossments should be neglected. Therefore, the effective area 'Ap' per meter and height of centre of area above bottom 'e' are usually based on tests. The plastic neutral axis ep is generally larger than e.

The simple plastic theory of flexure is used for analysis of these floors for checking the design at Limit State of collapse load. Eurocode assumes the equivalent ultimate stress of concrete in compression as 0.85(fck)cy/ϒc is the characteristics cylinder compression strength of concrete. However, IS 456:2000 uses an average stress of 0.36 (fck) cu accommodating the value of c and considering (fck) cu as Compressive force in concrete Ncf equal to steel yield force Npa.

**Shear Resistance of Composite Slab**

The shear resistance of composite slab largely depends on connection between profiled and deck concrete.

Shear resistance of composite slab consists of:

- Natural bond between concrete and steel due to adhesion
- Mechanical interlock provided by dimples on sheet and shear connectors
- Provision of end anchorage by shot fired pins or by welding studs when sheeting is made to rest on steel beams

**Design Considerations**

Composite floors are designed based on limit state design philosophy. Since IS 456:2000 is also based on limit state methods, the same has been followed wherever it is applicable.

The design should ensure an adequate degree of safety and serviceability of structure. The structure should therefore be checked for ultimate and serviceability limit states.

The main economy in using profiled deck is achieved due to speed in construction. Normally 2.5 to 4.0m spans can be handled without propping and spans in excess f 4m will require propping. The yield strength of decking steel is in the range of 220 to 460 $N/mm^2$. Though light – weight concrete is preferable both from reducing the effect of ponding deflection as well as increasing the fire resistance, the normal practice in India is to use concrete of grade M 20 to M 30.

The profiled deck depth normally available ranges from 40 to 85mm and the metal substrate thickness 0.7mm to 1.6mm. The normal span/depth values for continuous composite slab should be chosen to be less than 35. The overall depth of the composite slab should not be less than 90mm and thickness of concrete, $h_c$, shall not be less than 50mm.

**Profiled Steel Sheeting As Shuttering**

Verification is required for the profiled steel sheeting at the construction stage when it is acting as formwork for the wet concrete, construction loads and storage loads if any. While calculating the loads on the profiled sheet, increased depth of concrete due to deflection of the sheeting i.e., ponding effect has to be considered. Account should be taken of the effect of props, if any.

If the central deflection (δ) of the profiled deck in non-composite stage is less than l/325 or 20mm, whichever is smaller, then the ponding effect may be ignored in the design of profiled deck.

**Loads on Profiled Sheeting**

Construction loads, which include the weight of operatives, concreting plant and any impact, or vibration, that may occur during construction should be considered in the design. These loads should be arranged in such a way that they cause maximum bending moment and shear. In any area of 3m by 3m (or the span length, if less), in addition to weight of wet concrete, construction loads and weight of surplus concrete should be provided for by assuming a load of 1.5 kN/m$^2$. Over the remaining area a load of 0.75 kN/m$^2$ should be added to the weight of wet concrete.

**Composite slab**

The loads are applied in such a way that the load combination is most unfavorable. Load factors of 1.5 for both dead load and imposed load are employed in design calculations. Verification is required for the floor slab after composite behavior has commenced and any props have been removed.

Generally it is sufficient to consider the following load combinations in buildings mostly subject to uniformly distributed loads:

- Alternate spans carrying total factored loading due to imposed and dead loads. Other spans carrying only factored loading due to dead load.
- Any two adjacent spans carrying total factored load due o imposed and dead load and all other spans carrying only factord dead load.

**Diaphragm Action of Deck Slab**

Desk slab transmits in-plane loads for ensuring lateral stability of the building system. For this the deck slab is attached on all the four sides at spacing exceeding 600mm on either the beams or supporting walls. The diaphragm action is excellent if through deck welding is resorted to. The steel decking also provides lateral support to the steel beams it supports. However, beams running parallel to the decking are laterally supported only at transverse beam connections.

**Steps in the Design of Profiled Decking**

The following are the steps for design of profiled decking sheets:

(i) List the decking sheet data (Preferably from manufacturer's data)

(ii) List the loading

(iii) Design the profiled sheeting as shuttering

- Calculate the effective length of the span
- Compute factored moments and vertical shear
- Check adequacy for moment
- Check adequacy for vertical shear
- Check deflections

(iv) Design the composite slab – Generally the cross sectional area of the profiled decking that is needed for the construction stage provides more than sufficient reinforcement for the composite slab. So, the design of short span continuous slabs can be done as series of simply supported slabs and top longitudinal reinforcement is provided for cracking. However, long-span slabs are designed as continuous over supports.

- Calculate the effective of the span
- Compute factored moments and vertical shear

- Check adequacy for moment
- Check adequacy for vertical shear
- Check adequacy for longitudinal shear
- Check for serviceability, i.e. cracking above supports and deflections

### 2.1.3 COMPOSITE BEAMS

**Composite Action in Beams**

Composite beams, subjected mainly to bending, consist of steel section acting compositely with flange of reinforced concrete. To act together, mechanical shear connectors are provided to transmit the horizontal shear between the steel beam and the concrete slab, ignoring the effect of any bond between the two materials. These also resist uplift force acting at the steel concrete interface.

If there is no connection between concrete slab and steel beam at the interface, relative slip occurs between the steel section and the concrete slab when the beam is loaded. Thus, each component will act independently. With the help of a proper connection between concrete slab and steel beam at the interface, this slip can be minimized or even eliminated altogether. If slip at the interface is eliminated or drastically reduced, the slab on the extent to which slip is prevented. The degree of interaction depends mainly on the degree of shear connection used. Slip is zero at the mid-span and maximum at the support of a simply supported beam subjected to uniformly distributed load. Hence, shear is less in connectors located near the center and maximum in connectors located near the supports.

The connection is considered to be complete if the bending resistance, not the horizontal shear resistance, decides the resistance of composite beam. Complete or incomplete interaction between the concrete slab and steel section results in a more-stiff composite beam. Incomplete interaction arises when flexible connectors are used and slip (relative displacement) occurs at the steel concrete interface.

Composite beams are often designed under the assumption that the un-propped steel beam supports the weight of the structural steel and wet concrete plus construction loads. It may, therefore, be decided for reasons of economy to provide only sufficient connectors to develop enough composite action to support the loads applied afterwards. This approach results in considerably less number of connectors than are required to enable the maximum bending resistance of the composite beam to be reached. However, the use of such partial shear connection results in reduced resistance and stiffness.

**Degree of Interaction**

When no slip occurs between the concrete slab and the supporting steel beam, it is termed as full interaction. In other words, when the bending strength of a beam (local buckling arrested) does not increase with the addition of further connectors at the steel-concrete interface, it is considered that the complete shear connection has been achieved. In practice some slip will always occur and the term full interaction is used where it is considered that the effects of slip between the concrete flange and steel beam may be neglected in the design.

Partial interaction implies that slip occurs at the interface between the concrete flange and the steel beam, and hence it causes a discontinuity of strain that has to be taken into account in the analysis. Thus, in partial shear connection the number of connectors provided is less than that required to achieve complete shear connection. However, partial shear connection should not be considered as unsatisfactory for the purpose for which they are provided. Though not permissible as per Indian Standard, partial shear connection is of interest where the bending strength of the particular beam need not be fully utilized, for example where the size of the steel member is governed by the load carried by the steel beam only in an un-propped construction, or where the size of the member is calculated from service ability criteria point of view rather than the strength.

**Basic Design Considerations**

The analysis of composite section is made using Limit State of collapse method. IS: 11384 – 1985 Code deals with the design and constructions of only simply supported composite beams. Therefore, the method of design suggested in EC 4 is also referred along with IS: 11384.

## 2.1.4 ENCASED COLUMNS

**Introduction**

A composite member subjected mainly to compression and bending is called as composite column. At present, there is no Indian Standard covering the design of composite columns. The method of design suggested in this chapter largely follows EC4, which incorporates the latest research on composite construction.

Indian Standards for composite construction (IS: 11384-1985) does not make any specific reference to composite columns. The provisions contained in IS:456 – 2000 are often invoked for design of composite structures. Extension of IS: 456 – 2000 to composite columns will result in the following equation:

$$\mathbf{P_p = A_a P_y + A_c\, P_{CR} + A_S\, P_{sk}}$$

Where,

$$\mathbf{P_y = 0.8\, f_y;\ p_{ck} = 0.4(f_{ck})_{cu}\ and\ p_{sk} = 0.67\, f_y}$$

The concrete and steel are combined in such a fashion that the advantages of both the material are utilized effectively in composite column. There are many advantages associated with the use of steel-concrete composite columns: small cross-sections, for example, can be designed to withstand high loads; similarly, sections with different resistances, but identical external dimensions, can be produced by varying steel area, concrete strength and additional reinforcement. Thus the outer dimension of a column can be held constant over a number of floors in a building, simplifying architectural detailing. Also steel-concrete composite members help to improve the fire resistance. A steel-concrete composite column consists of either a concrete encased hot-rolled steel section or a concrete filled tubular section of hot-rolled steel and is generally deed as a load-bearing member in a composite framed structure.

In a composition column both the steel and concrete would resists the external loading by interacting together by bond and friction. Additional reinforcement in the concrete encasement prevents excessive spalling of concrete both under normal load and fire conditions.

During construction, bare steel sections support the initial construction loads, including the weight of structure during construction. Concrete is later cast around the steel section, or filled inside the tubular sections. The lighter weight and higher strength of steel permit the use of smaller and lighter foundations. The subsequent concrete addition enables the building frame to easily limit the sway and lateral deflections.

By employing composite columns, the speed of construction can be increased and it is possible to erect the structures in most efficient manner with significant economic advantages over either pure structural steel/or reinforced concrete alternatives.

Apart from speed and economy, the following other important advantages can be achieved.

- Increased strength for a given cross sectional dimension.
- Increased stiffness, leading to reduced slenderness and increased buckling resistance.
- Fire resistance in the case of concrete encased columns in much better.
- Identical cross sections with different load and moment resistances can be produced by varying steel thickness. the concrete strength and reinforcement. This allows the outer dimensions of a column to be held constant over a number of floors in a building, thus simplifying the construction and architectural detailing.
- Erection of high rise building in an extremely efficient manner.
- Formwork is not required for concrete filled tubular sections.

There is quite a vertical spread of construction activity carried out simultaneously at any one time, with numerous trades working simultaneously. For example

- One group of workers can be erecting the steel beams and columns for one or two storey at the top of frame.
- Two or three storey below, another group of workers may be fixing the metal decking for the floors.

- A few storey below, another group may be concreting the floors.
- As we go down the building, another group may be tying the column reinforcing bars in cases.
- Yet another group below them may be fixing the formwork, placing the concrete into the column moulds etc.

**Composite Column Design**

Design of composite column is based on limit state method. Euro code 4 is generally followed for composite column design, as there is no Indian Standard covering composite columns.

For structural adequacy, the internal forces and moments resulting from the most unfavorable load combination should not exceed the design resistance of the composite cross-sections. While local buckling of the steel sections may be eliminated, the reduction in the compression resistance of the composite column due to overall buckling should definitely be allowed for, together with the effects of residual stresses and initial imperfections. Moreover, the second order effects in slender columns as well as the effect of creep and shrinkage of concrete under long-term loading must be considered, if they are significant. The reduction in flexural stiffness due to cracking of the concrete in the tension are should also be considered.

Isolated symmetric columns having uniform cross sections in braced or non-sway frames may be designed by the simplified design method, which adopts the European buckling curves for steel columns. However, this method cannot be applied to sway columns.

When a sufficiently stiff frame is subjected to in-plane horizontal forces, the additional internal forces and moments due to the consequent horizontal displacement of its nodes can be neglected, and the frame is classed as "non-sway".

**Fire resistance**

Composite columns were actually developed for their inherent high resistance. Composite columns are usually designed in the normal or `cool' state and then checked under fire conditions. Additional reinforcement is sometimes required to achieve the target fire resistance. Some general rules on the structural performance of composite columns in fire are summarized as follows.

- The fire resistance of composite columns with fully concrete steel sections may be treated in the same way as reinforced concrete columns. An appropriate concrete cover insulates the steel and light reinforcement is also required in order to maintain the integrity of the concrete cover. In such cases, two-hour fire resistance can usually be achieved with the minimum concrete cover of 40mm.

- For composite columns with partially concrete encased steel sections, the structural performance of the columns is very different in fire, as the flanges of the steel sections are exposed and less concrete acts as a `heat shield'. In general, a fire resistance of up to one hour can be achieved if the strength of concrete is neglected in normal design. Additional reinforcement is often required to achieve more than one-hour fire resistance.

- For concrete filled tubular sections subjected to fire, the steel sections are exposed to direct heating while the concrete core behaves as `heal sink'. In general, sufficient redistribution of stress occurs between the hot steel sections and the relatively cool concrete core, so that a fire resistance of one hour can usually be achieved.

Steel fiber reinforcement is also effective in improving the fire resistance of a concrete filled column. It is also a practice in India to wrap the column with fibrocement to increase the fire rating.

# CHAPTER 3

# LOAD CALCULATION

## 3.1 DEAD LOAD

Dead load was taken as per IS 875 (Part I)-1987

At any floor level

(Assuming thickness of slab = 125 mm)

Load from slab = 3.125 $KN/m^2$ = 0.125 x 25

Partitions = 1.5 KN/ $m^2$

Floor finishes = 0.05 x 24 = 1.2 KN/ $m^2$

Weight of Metal deck = 0.15 KN/ $m^2$

Weight of duct & Plastering = 0.8 $KN/m^2$

= 6.775 $KN/m^2$

Total = 6.8 $KN/m^2$

At roof level

Load from slab = 0.125 x 25= 3.125 $N/m^2$

Weight of Metal deck = 0.15 $KN/m^2$

Screed concrete 50mm thick = 0.05 x 24

= 1.2 $KN/m^2$

False ceiling, ducts etc., = 1.0 $KN/m^2$

Total = 5.475 $KN/m^2$

= 5.5 $KN/m^2$

## 3.2 LIVE LOAD

Live load was taken as per IS : 875 (Part II)-1987

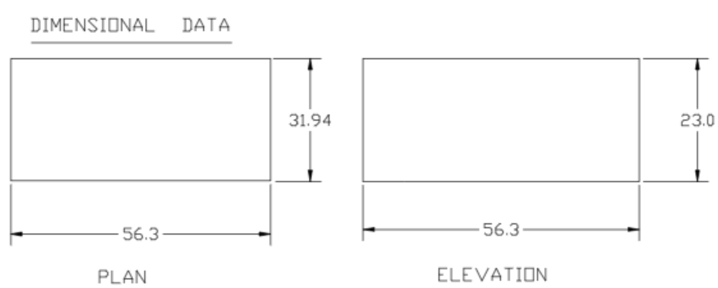
Live load was found to be 4.00 $KN/m^2$ for Institutional Buildings.

### 3.3 WIND LOAD

Wind load was taken as per IS : 875 (Part III) -1987

Location : Chennai

Basic wind speed = 50m/s




**3.3.i PLAN & ELEVATION**

$K_1$ = Risk factor

Design life = 50 yrs

$K_1 = 1.0$

$K_2$ = Terrain, Height & Structure size factor

Terrain type - Category 3

Structure - Class B

Height - 21.9 m + 0.6 $\approx$ 23m

$K_2$ - 0.99

$K_3$ – Topography Factor

Assuming that upwind slope < 3 degrees

Topography is not significant

$K_3 = 1.0$

$V_z = 0.99 \times 1.0 \times 1.0 \times 50$

$= 49.5$ m/s $\approx$ 50m/s

Design Wind Pressure = $0.6\ V_z^2$

$= 0.6 \times 50^2$

$= 1500\ N/m^2$

$= 1.5\ KN/m^2$

<u>Design Wind Loads</u>

WIND LOAD, $F = P_z \times A_c \times C_f$

Where

$P_z$ = Design pressure at height z

$A_c$ = Effective frontal area at height z

$C_f$ = Force coefficient for the building

## 3.3.a WIND LOADS-PARALLEL TO SHORTER DIRECTION

| Floor | GF | 1ST | 2ND | 3RD | 4TH | 5TH |
|---|---|---|---|---|---|---|
| H | 4.51 | 8.17 | 11.83 | 15.55 | 19.15 | 22.81 |
| $A_c$ | 10.125 | 16.47 | 16.47 | 16.47 | 16.47 | 8.23 |
| $C_f$ | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| $P_z$ | 0.61 | 1.11 | 1.40 | 1.51 | 1.60 | 1.84 |
| F | 6.17 | 18.28 | 23.05 | 24.86 | 26.35 | 15.14 |

### 3.3.b WIND LOADS-PARALLEL TO LONGER DIRECTION

| Floor | GF | $1^{ST}$ | $2^{ND}$ | $3^{RD}$ | $4^{TH}$ | $5^{TH}$ |
|---|---|---|---|---|---|---|
| H | 4.51 | 8.17 | 11.83 | 15.55 | 19.15 | 22.81 |
| $A_c$ | 20.52 | 40.95 | 40.95 | 40.95 | 40.95 | 16.65 |
| $C_f$ | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| $P_z$ | 0.61 | 1.11 | 1.40 | 1.51 | 1.60 | 1.84 |
| F | 12.52 | 45.45 | 57.33 | 61.83 | 65.52 | 30.636 |

### 3.4 SEISMIC LOAD

Seismic load were taken as per IS:1893(Part I) : 2002

The seismic coefficient method is adopted.

1. The city of Chennai falls under Zone III

   Zone Factor = 0.16

   Importance Factor = 1.0

   Response Reduction Factor = 5

   Approximate Fundamental Period = $0.09\ H / D^{1/2}$

   $= 0.09 \times 23 / 56.3^{1/2}$

   $= 0.27s$

2. Base Shear

   $V_B = K\ C\alpha_h W$

   $V_B$ = Base shear

   Performance Factor = 1.3

   C = a coefficient which depends upon the fundamental time periods

   $\beta$ = A factor depending upon the soil foundation system = 1.2

   $\alpha_0$ = basic horizontal seismic coefficient = 0.04

$\alpha_h$ = design seismic Coefficient

$= \beta\ \alpha_0\ I = 1.2 \times 0.04 \times 1.5 = 0.072$

Total Load on any floor = 6.8 x 56 x 32 + 0.25 x 4 x 56 x 32

= 13977.6 kN

Total load on the roof = 5.5 x 56 x 32

= 9856 kN

Total Load = 6 x 13977.6 + 5.5 x 56 x 32

= 93721.6 kN

$V_B$ = 1.3 x 1.0 x 0.072 x 93721.6

= 8772.34 kN

3. Calculation of Lateral Forces

$Q_i = V_B\ W_i h_i^2 / \sum W_i h_i^2$

$Q_i$ = Lateral Force at floor i

$W_i$ = Load on the floor i

$h_i$ = Height measured from the base of the building to the floor i

**3.3.c** CALCULATION OF LATERAL FORCES

| Floor level | $h_i$ (m) | $W_i$ (kN) | $W_i h_i^2$ | $Q_i = V_B\ W_i h_i^2/\sum W_i h_i$ | $V_j = \sum Q_i$ |
|---|---|---|---|---|---|
| G floor | 3.66 | 13977.6 | $0.187 \times 10^6$ | 73.77 | 8770.38 |
| 1st floor | 7.32 | 13977.6 | $0.749 \times 10^6$ | 295.48 | 8696.6 |
| 2nd floor | 10.98 | 13977.6 | $1.685 \times 10^6$ | 664.72 | 8401.13 |
| 3rd floor | 14.64 | 13977.6 | $2.99 \times 10^6$ | 1179.53 | 7736.4 |
| 4th floor | 18.3 | 13977.6 | $4.681 \times 10^6$ | 1846.62 | 6556.88 |
| 5th floor | 21.96 | 13977.6 | $6.74 \times 10^6$ | 2658.9 | 4716.26 |
| Roof | 23.0 | 9856 | $5.2 \times 10^6$ | 2051.4 | 2051.4 |
| | | | $22.236 \times 10^6$ | | |

# CHAPTER 4

# ANALYSIS

- ✓ Analysis was done using STAAD-Pro III.
- ✓ Analysis was done assuming that the building is a concrete building.
- ✓ 2D analysis was done for two cases:-
  1. Frame along shorter direction
  2. Frame along longer direction
- ✓ Footing was idealized as fixed support.
- ✓ The load cases adopted are dead load and live load, wind load and the seismic load
- ✓ Analysis was done for the load combinations given below:
  1. Dead load + live load
  2. Dead load + live load + wind load in (+ve) x – direction
  3. Dead load + live load + wind load in ( - ve) x – direction
  4. Dead load + live load +earthquake load in ( + ve) x – direction
  5. Dead load + live load +earthquake load in ( - ve) x – direction
- ✓ The edit info file containing the program for analysis is given below:

**Analysis of an intermediate frame in shorter direction**

- **STAAD PLANE HITECH 1**

```
START JOB INFORMATION
ENGINEER DATE 04-Mar-05
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
```

JOINT COORDINATES

1 0 0 0; 2 9.14 0 0; 3 18.28 0 0; 4 0 1.15 0; 5 9.14 1.15 0; 6 18.28 1.15 0;

7 0 5.65 0; 8 9.14 5.65 0; 9 18.28 5.65 0; 10 0 9.31 0; 11 9.14 9.31 0;

12 18.28 9.31 0; 13 0 12.97 0; 14 9.14 12.97 0; 15 18.28 12.97 0; 16 0 16.63 0;

17 9.14 16.63 0; 18 18.28 16.63 0; 19 0 20.29 0; 20 9.14 20.29 0;

21 18.28 20.29 0; 22 0 23.95 0; 23 9.14 23.95 0; 24 18.28 23.95 0;

MEMBER INCIDENCES

1 4 5; 2 5 6; 3 7 8; 4 8 9; 5 10 11; 6 11 12; 7 13 14; 8 14 15; 9 16 17;

10 17 18; 11 19 20; 12 20 21; 13 22 23; 14 23 24; 15 1 4; 16 2 5; 17 3 6;

18 4 7; 19 5 8; 20 6 9; 21 7 10; 22 8 11; 23 9 12; 24 10 13; 25 11 14;

26 12 15; 27 13 16; 28 14 17; 29 15 18; 30 16 19; 31 17 20; 32 18 21; 33 19 22;

34 20 23; 35 21 24;

DEFINE MATERIAL START

ISOTROPIC CONCRETE

E 2.17185e+007

POISSON 0.17

DENSITY 23.5616

ALPHA 5.5e-006

DAMP 0.05

END DEFINE MATERIAL

```
CONSTANTS
MATERIAL CONCRETE MEMB 1 TO 35
MEMBER PROPERTY INDIAN
1 TO 35 PRIS YD 0.15 ZD 0.15
SUPPORTS
1 TO 3 FIXED
LOAD 1 DL+LL
MEMBER LOAD
1 TO 14 UNI GY -32.4
1 TO 14 CON GY -93.15 3.05 6.1
1 TO 14 CON GY -93.15 6.1 3.05
1 2 UNI GY -9.79
LOAD 2 WIND LOAD
JOINT LOAD
7 FX 6.17
10 FX 18.28
13 FX 23.05
16 FX 24.86
19 FX 26.35
22 FX 15.14
LOAD 3 SEISMIC LOAD
JOINT LOAD
4 FX 9.22
7 FX 36.935
10 FX 83.09
13 FX 147.44
16 FX 230.82
19 FX 332.36
22 FX 256.42
```

LOAD COMB 4 DL+LL1

1 1.5

LOAD COMB 5 DL+LL+WL

1 1.2 2 1.2

LOAD COMB 6 DL+LL+EQL

1 1.2 3 1.2

LOAD COMB 7 DL+LL-WL

1 1.2 2 -1.2

LOAD COMB 8 DL+LL-EQL

1 1.2 3 -1.2

PERFORM ANALYSIS

PRINT ANALYSIS RESULTS

PERFORM ANALYSIS PRINT ALL

**FINISH**

**Analysis of an intermediate frame in longer direction**

- **STAAD PLANE HITECH 2**

START JOB INFORMATION

ENGINEER DATE 05-Mar-05

END JOB INFORMATION

INPUT WIDTH 79

UNIT METER KN

JOINT COORDINATES

1 0 0 0; 2 4.5 0 0; 3 9 0 0; 4 13.5 0 0; 5 18 0 0; 6 22.5 0 0; 7 27 0 0;

8 31.5 0 0; 9 0 1.15 0; 10 4.5 1.15 0; 11 9 1.15 0; 12 13.5 1.15 0;

13 18 1.15 0; 14 22.5 1.15 0; 15 27 1.15 0; 16 31.5 1.15 0;
17 0 5.65 0;
18 4.5 5.65 0; 19 9 5.65 0; 20 13.5 5.65 0; 21 18 5.65 0; 22
22.5 5.65 0;
23 27 5.65 0; 24 31.5 5.65 0; 25 0 9.31 0; 26 4.5 9.31 0; 27 9
9.31 0;
28 13.5 9.31 0; 29 18 9.31 0; 30 22.5 9.31 0; 31 27 9.31 0;
32 31.5 9.31 0;
33 0 12.97 0; 34 4.5 12.97 0; 35 9 12.97 0; 36 13.5 12.97 0;
37 18 12.97 0;
38 22.5 12.97 0; 39 27 12.97 0; 40 31.5 12.97 0; 41 0 16.63
0; 42 4.5 16.63 0;
43 9 16.63 0; 44 13.5 16.63 0; 45 18 16.63 0; 46 22.5 16.63
0; 47 27 16.63 0;
48 31.5 16.63 0; 49 0 20.29 0; 50 4.5 20.29 0; 51 9 20.29 0;
52 13.5 20.29 0;
53 18 20.29 0; 54 22.5 20.29 0; 55 27 20.29 0; 56 31.5 20.29
0; 57 0 23.95 0;
58 4.5 23.95 0; 59 9 23.95 0; 60 13.5 23.95 0; 61 18 23.95 0;
62 22.5 23.95 0;
63 27 23.95 0; 64 31.5 23.95 0;
MEMBER INCIDENCES
1 9 10; 2 10 11; 3 11 12; 4 12 13; 5 13 14; 6 14 15; 7 15 16;
8 17 18; 9 18 19;
10 19 20; 11 20 21; 12 21 22; 13 22 23; 14 23 24; 15 25 26;
16 26 27; 17 27 28;
18 28 29; 19 29 30; 20 30 31; 21 31 32; 22 33 34; 23 34 35;
24 35 36; 25 36 37;

26 37 38; 27 38 39; 28 39 40; 29 41 42; 30 42 43; 31 43 44; 32 44 45; 33 45 46;

34 46 47; 35 47 48; 36 49 50; 37 50 51; 38 51 52; 39 52 53; 40 53 54; 41 54 55;

42 55 56; 43 57 58; 44 58 59; 45 59 60; 46 60 61; 47 61 62; 48 62 63; 49 63 64;

50 1 9; 51 2 10; 52 3 11; 53 4 12; 54 5 13; 55 6 14; 56 7 15; 57 8 16; 58 9 17;

59 10 18; 60 11 19; 61 12 20; 62 13 21; 63 14 22; 64 15 23; 65 16 24; 66 17 25;

67 18 26; 68 19 27; 69 20 28; 70 21 29; 71 22 30; 72 23 31; 73 24 32; 74 25 33;

75 26 34; 76 27 35; 77 28 36; 78 29 37; 79 30 38; 80 31 39; 81 32 40; 82 33 41;

83 34 42; 84 35 43; 85 36 44; 86 37 45; 87 38 46; 88 39 47; 89 40 48; 90 41 49;

91 42 50; 92 43 51; 93 44 52; 94 45 53; 95 46 54; 96 47 55; 97 48 56; 98 49 57;

99 50 58; 100 51 59; 101 52 60; 102 53 61; 103 54 62; 104 55 63; 105 56 64;

DEFINE MATERIAL START

ISOTROPIC CONCRETE

E 2.17185e+007

POISSON 0.17

DENSITY 23.5616

ALPHA 5.5e-006

DAMP 0.05

END DEFINE MATERIAL

CONSTANTS

```
MATERIAL CONCRETE MEMB 1 TO 105
MEMBER PROPERTY INDIAN
1 TO 105 PRIS YD 0.15 ZD 0.15
SUPPORTS
1 TO 8 FIXED
LOAD 1 DL+LL
MEMBER LOAD
1 TO 49 UNI GY -2.07
1 TO 7 UNI GY -9.79
LOAD 2 WIND LOAD
JOINT LOAD
41 FX 61.83
49 FX 65.52
57 FX 30.636
33 FX 57.33
25 FX 45.45
17 FX 12.52
LOAD 3 SEISMIC LOAD
JOINT LOAD
9 FX 9.22
17 FX 36.935
25 FX 83.09
33 FX 147.44
41 FX 230.82
49 FX 332.36
57 FX 256.42
LOAD COMB 4 DL+LL
1 1.5
LOAD COMB 5 DL+LL+WL +X dir
```

1 1.2 2 1.2

LOAD COMB 6 DL+LL+WL -X dir

1 1.2 2 -1.2

LOAD COMB 7 DL+LL+EQL + X dir

1 1.2 3 1.2

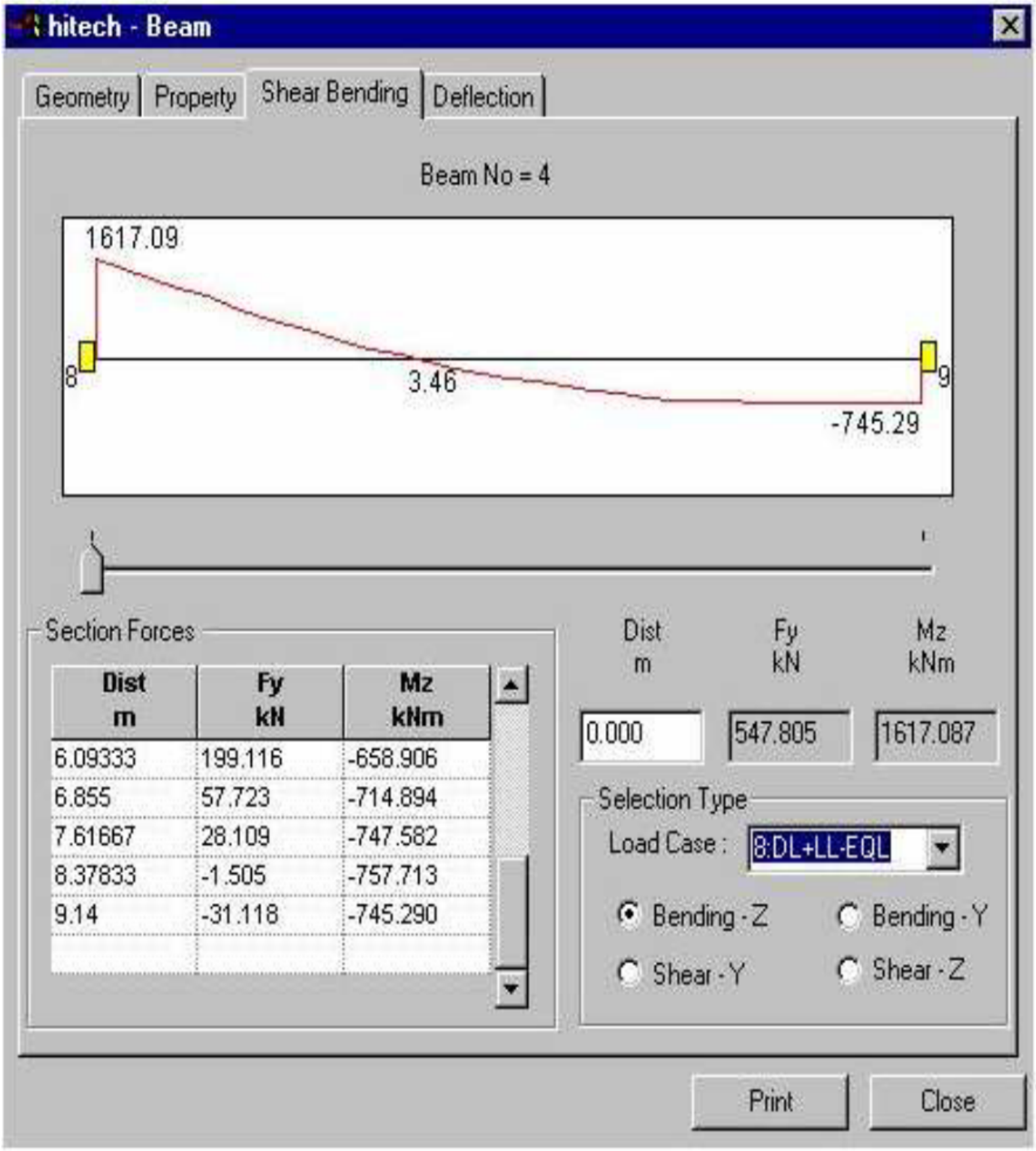
LOAD COMB 8 dl+ll-EQL  - X dir

1 1.2 3 -1.2

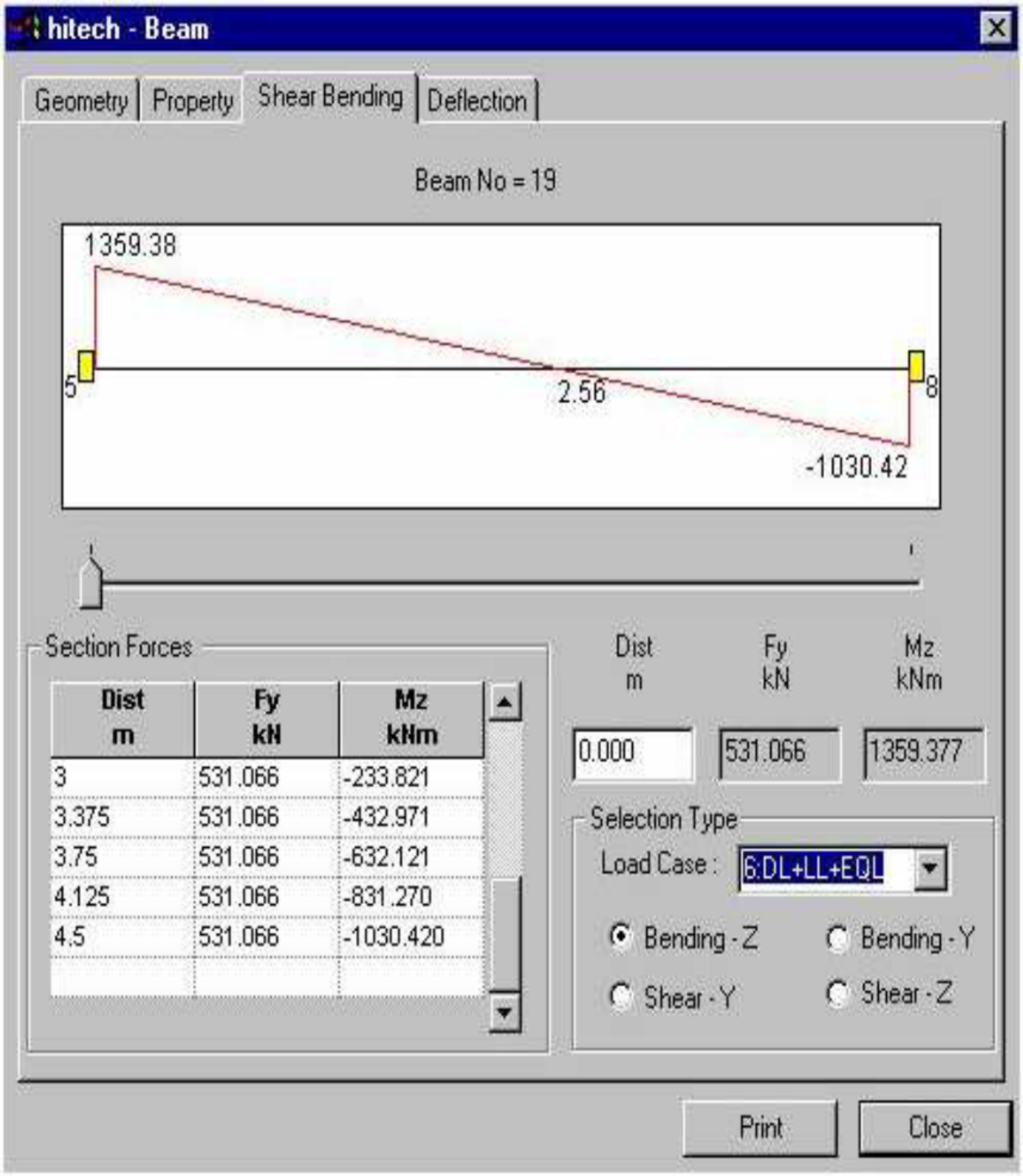
PERFORM ANALYSIS PRINT ALL

**FINISH**

## 4.i BENDING MOMENT DIAGRAM FOR BEAM



## 4.ii BENDING MOMENT DIAGRAM FOR COLUMN



# CHAPTER 5

# DESIGN OF ELEMENTS (COMPOSITE)

## 5.1 DESIGN OF SLAB

Slabs are designed as per BS:5950 :Part 4

Effective Span = 3.0m

Total Depth of the slab = 125 mm

Live load = $4kN/m^2$

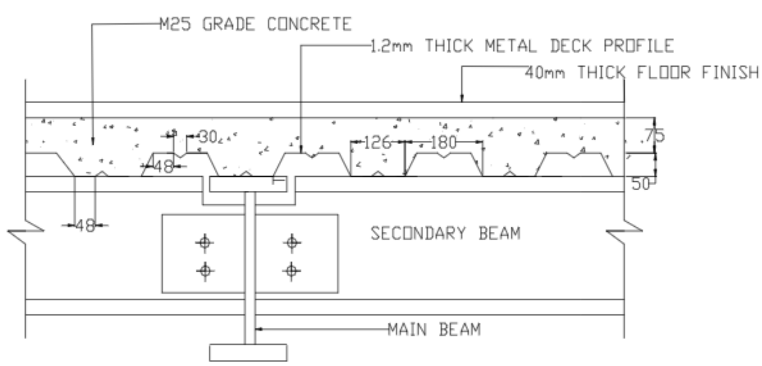
Grade of concrete = M25

Density of concrete (dry) = 24 kN/ $m^2$

Density of concrete (wet) = 25 kN/ $m^2$

$t_s$ = 125 mm

$f_{yp}$ = 345 $N/mm^2$




PROFILE DECKING

**5.1.i CROSS SECTION OF PROFILED DECK SLAB**

a = 27 mm

b = 48 mm

c = 30 mm

d = 50 mm

e = 65.82 mm

f = 48 mm

h = 75 mm

Spacing of trough = 126 + 180 = 306 mm

1. <u>LOAD DATA</u>

<u>At any floor level</u>

Load from slab = 0.125 x 25 = 3.125 kN/m$^2$

Total load = 6.8 kN/m$^2$

<u>At roof level</u>

Total load = 5.5x1.5 = 8.25kN/ m$^2$...

Imposed load = 4.00 kN/ m$^2$........................

Factored load = 6 kN/ m$^2$.............

Construction load = 1.5 kN/ m$^2$...

2. <u>AT CONSTRUCTION STAGE</u>

Total self weight = 3.125+0.15

= 3.275 kN/ m$^2$

Ultimate design moment at construction stage considering

A load factor of 1.4.

$$M_u = \frac{1.4(3.275 + 1.5)3^2}{8} = 7.52 \text{ kN.m}$$

(FOS = 1.4 as per BS:5950)

Check if $\frac{b}{t} \leq \frac{560}{\sqrt{f_y}}$

$$\frac{b}{t} = \frac{48}{1.2} = 40$$

$$\frac{560}{\sqrt{f_y}} = 30.15$$

$\frac{b}{t} > 30.15$

$$b_{es} = \frac{828t}{\sqrt{f_y}}\left(1 - \frac{181t}{b\sqrt{f_y}}\right)$$

$$b_{es} = \frac{828 \times 1.2}{\sqrt{345}}\left(1 - \frac{181 \times 1.2}{48\sqrt{250}}\right)$$

$= 40.46 < 48\text{mm}$

b = 48mm

Neutral axis position = 27.46mm

I = $275.13 \times 10^3$ $mm^4$ /trough

= $899.12 \times 10^3$ $mm^4$

d/t = (58.8-27.46)/1.2 = 26.12

$Z_e$ = $899.12 \times 10^3$ / (58.8-27.46) = $25.15 \times 10^3$ $mm^3$ / m

Elastic moment of resistance of section, $M_e = Z_e \; x \; 0.93 x f_y$ = 9.03 kN.m

After construction,

Bending moment = $\frac{3.275 \times 3^2}{8}$

= 3.68 kN.m

Equivalent stress in compression plate,

$\sigma = \frac{3.68 \times 345}{9.711}$ = 130.89 $N/mm^2$

$\frac{560}{\sqrt{f_y}} = \frac{560}{\sqrt{130.89}}$ = 48.94> b

$b_{es}$ = 54.36 >b

Deflection of soffit of simply supported beam,

$$\delta = \frac{5 \times 3.275 \times 3000^4}{2 \times 10^5 \times 899.12 \times 1000}$$

= 18.7 mm <(span/150)

3. COMPOSITE CONDITION

Area of deck, $A_p$ = ( 126 + 65.8 ) 2 x 1.2 =1504.31 sq.m / 3

Tensile resistance = $(0.93 \times 345 \times 1504.31 \times 10^3)$

= 482.65 kN

$X_c = \frac{482.65}{0.4 \times 25}$

= 48.27 mm

Plastic Moment of resistance of Composite Section,

$M_p = 482.66 \times (125 - 48 - 48.21/2) \times 10^{-3}$

= 25.52 kN.m

$M_u = \frac{(1.6 \times 4 + 1.4 \times 6.8) \times 3^2}{8}$ (1.6 x 4 + 1.4 x 6.8) x 3 x 3/8

= 17.91 kN.m

$V_u = \frac{B_d d_s}{1.25 L_v} \left[ \frac{m_r A_p}{B_s L_v} + k_r \sqrt{f_{ck}} \right]$

$B_s d_s = Ac \left[ 65 + \frac{60}{2} \right] \times 10^3 = 95000$ sq.mm.

$D_c$ = 95 mm

$A_p$ = 1504.31 $mm^2$

$L_v = \frac{3000}{4}$ = 750 mm

Typical values of empirical constants are:

$m_r$ = 130

$k_r$ = 0.004

$V_u$ = 28.45 kN

Applied S.F. = $4 \times \frac{M_u}{L}$

$= 4 \times \frac{17.91}{3} = 23.88 < V_u$

Shear stress $= \frac{23.88}{95} = 0.25$ N / $mm^2$

4. CHECK FOR DEFLECTION

   Properties of Cracked section,

   $I = 8907.5\ mm^4$ /mm width

   Service load on composite slab = 4kN/sq.m

   $$\delta = \frac{5\times4\times3000^4}{384\times2\times10^5\times8907.5}$$

   $= 2.31$ mm

   $3000/615 = 4.9 > \delta$

   Therefore ,safe.

## 5.2 DESIGN OF BEAMS

Secondary beams of ISMB 350 are used at a spacing of 3.00m in the shorter direction.

The beams are designed as per BS: 5950 Part III. The beam design shown below is the design of the beam for the first and second floors. The beam is designed as a continuous beam .The Bending moments and shear force were obtained from STAAD analysis

Maximum positive B.M $M_{u(+ve)}$= 1617.087 kN.m

Maximum negative B.M $M_{u(-ve)}$= 980.359 kN.m

Maximum S.F $V_u$= 547.085 kN.

DESIGN CALCULATIONS

1. INITIAL SELECTION OF BEAM SIZE

   Assume $\frac{Span}{Depth} = 22$

   Depth of the composite section = $\frac{9142}{22} = 415.3 \approx 450$mm

   Let us take ISHB 450@92.5kg/m with plates of thickness 20mm on both the flanges. The breadth of each flange is 350mm.

2. SECTION PROPERTIES

$T_f = 13.7$ mm

$B_f = 250$ mm

$D = 450$ mm

$t_w = 11.3$ mm

$B_p = 350$ mm

$t_p = 20$ mm

Total Depth of the section = 490 mm

$A_a = 25789\ mm^2$

3. CLASSIFICATION OF COMPOSITE SECTION

$$e = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

$$0.5\frac{B}{T} = 0.5\frac{350}{33.7} = 5.19 < 8.19\ e$$

$$\frac{d}{T} = \frac{(490 - 2 \times 33.7)}{11.3} = 37.4 < 83\ e$$

Therefore the section is a plastic section.

4. ULTIMATE LIMIT STAGE

Construction Stage

- Plastic Moment Resistance of the steel section,

$Z_{px} = 1.14\ Z_x$

$= 1.14 \times 4778.6 \times 1000 = 5447.62 \times 10^3 mm^3$

$M_{ap} = 0.87\ f_y\ Z_{px}$

$= 0.87 \times 250 \times 5447.62 \times 10^3$

$= 1184.9 kN.m <$ M(-ve)

- Plastic Shear Resistance

$V_p = 0.6\ Dt_w \times 0.87\ f_y$

$= 0.6 \times 490 \times 11.3 \times 0.87 \times 250$

$= 722.5kN > V_u$

Therefore, safe.

5. COMPOSITE STAGE

Maximum Negative Bending Moment

i. Effective width of the concrete flange

$$B_{eff} = \frac{l_o}{4}$$

$$= \frac{1(0.25(l_1 + l_2))}{4}$$

$$= \frac{1(0.25(9142 \times 2))}{4}$$

$$= 1142.75\ mm^2$$

Let us provide 12mm diameter bars at 100 mm c/c

$A_{st} = 1050mm^2$

ii. Location of Neutral Axis

$F_a = A_a \text{ x } 0.87 \text{ x } f_y$

$= 25789 \text{ x } 0.87 \text{ x } 250$

$=$ **5609.11 kN**

$F_s = 0.87 \text{ x } f_y \text{ x } A_{st}$

$= 0.87 \text{ x } 415 \text{ x } 1050$

$=$ **379 kN** $< F_a$

Let $x$ be the distance of the plastic N.A. from the flange of the beam

$2F_{a1} + F_s = F_a$

$F_{a1} = 0.87\ f_y\ (x \text{ x } 11.3 + (350 - 11.3)\ 20 + (250 - 11.3)\ 13.7)$

$= 217.4\ (11.3\ x + 11414.19)$

$2 \text{ x } 217.4\ (11.3\ x + 11414.19) + 379 \text{ x } 1000 = 5609.11 \text{ x } 10^3$

$x = 53.898mm$

$\underline{\mathbf{F_{a1} = 2615.05kN}}$

Taking moments about the centre of steel,

$2F_{a1}$ x (110 + 53.898/2) + $F_a$ (490 / 2 + 110 )

= 2 x 2615.05 x 136.95 + 5609.11 x 355

= 2707.496kN.m >980.359kN.m

$$X_c = \frac{F_s}{t_w \times 2 \times 0.87 \times f_y}$$

$$= \frac{379 \times 1000}{11.3 \times 2 \times 0.87 \times 250}$$

= 77.103mm

$$Z = \frac{D}{2} + d_s - \frac{X_c}{2}$$

$$= \frac{490}{2} + 125 - \frac{77.103}{2}$$

= 331.45 mm

$$Z_a = \frac{\left(\frac{D}{2} - X_c\right)}{2} + \frac{\left(\frac{D}{2} + X_c\right)}{2}$$

$$= \frac{490}{2}$$

= 245 mm

Taking moments about the centre of compression,

$M_p = F_s \, x \, Z + F_a \, x \, Z_a$

= 379 x 1000 + 331.45 + 5609.11 x 1000 x 245

= **1499.85kN.m** >980.46kN.m

Therefore the design is safe for negative B.M,

Maximum Positive Bending Moment

i. Effective width of the concrete flange

$$B_{eff} = \frac{l_o}{4}$$

$$= \frac{0.8 \times 9142}{4}$$

= 1828.4mm

ii. Location Of Neural Axis

Assuming that the N.A lies in the slab,

$0.36 \text{ x } f_{ck} \text{ x } b_{eff} \text{ x } x_u = 0.87 f_y A_s$

$x_u = \frac{5609.11 \times 1000}{1828.4 \times 0.36 \times 25}$

= 350.60 mm

Therefore the N.A. does not lie in the slab.

Let us assume that P.N.A lies at a distance $x$ from the flange of the beam.

$F_{cc} + 2 F_{a1} = F_a$

0.36 x 25 x 1828.4 x 125

+ 2 x 0.87 x 250 (20 (350-11.3 )

+ (250 – 11.3 )$x$ = 5609.11 kN.m

$x$ = 5.83mm

$x_u$ = 25.83mm

$M_p = F_a (0.5D + d_s - 0.42 x_u)$

= 5609.11 x 1000 ( 490/2 +125 -0.42 25.83 )

= **2014.52 kN.m**

>1617.087 kN.m

Therefore the design is safe for maximum positive B.M.

6. CHECK FOR VERTICAL SHEAR AND BENDING MOMENT AND SHEAR FORCE INTERACTION

$V_u$.= 547.085 kN

$V_p = 0.87 f_y \text{ x } 0.6\ d\ t_w$

= 0.87 x 250 x 0.6 x 490 x 11.3

= 722.578 kN >547.085 kN

Therefore ,design is safe.

Bending Moment and Vertical Shear interaction can be neglected if

$V_u < 0.5\ V_p$

$V_u > 0.5\ (722.578)$

Hence, interaction.

For the design to be safe,

$$M_u < M_f + (M_p - M_f)\left[1 - \left(\frac{2V_u}{V_p} - 1\right)^2\right]$$

$M_f$ = Plastic moment of resistance of the flange alone

$= 0.87\ f_y \text{ x } B \text{ x } T \text{ x } 2 \text{ x } 0.5d$

$= 0.87 \text{ x } 250 \text{ x } 250 \text{ x } 33.7 \text{ x } 2 \text{ x } 0.5 \text{ x } 422.6$

= 774.39 kN.m

$M_p$ = 1499.85 kN.m

$$774.39 + (1499.39 - 774.39)\left[1 - \left(\frac{2 \times 547.085}{722.578} - 1\right)^2\right] = 1307.99$$

$M_u <$ **<u>1307.99 kN.m</u>**

Therefore, the design is safe

<u>Check for shear buckling</u>

$$\frac{d}{t_w} = \frac{(490.2 - 2 \times 33.7)}{11.3} = 37.39 < 67e$$

hence,safe

7. <u>CHECK FOR DEFLECTION</u>

$$\delta = \frac{5 \times w \times l^4}{384 \times E_s \times I}$$

$$= \frac{5 \times 32.4 \times 9142^4}{384 \times 2 \times 10^5 \times 1170.76 \times 10^6}$$

= **<u>12.48 mm</u>**

$< (l/325) = 9142 / 325 = 28$ mm

hence, safe

8. <u>DESIGN OF SHEAR CONNECTORS</u>

<u>Longitudinal Shear Force</u>

- Between simple end support and point of maximum +ve moment

  Length = 0.4 $l$ = 0.4 x 9142 = 3656.8mm

  $V_v = F_a$ = internal resistance

  Assuming full shear connection, let us provide 22mm diameter studs with 100mm height

  P = 85 kN

  $$\text{No.of shear connectors} = \frac{F_a}{85}$$

  $$= \frac{5609.11}{85} = 70$$

  $$\text{Spacing} = \frac{3658.8}{70} = 55\text{mm}$$

- Between point of maximum +ve moment and internal support

  Length = $l$ – 0.4 $l$ = 9142 – 3656.8 = 5485.4mm

  $V_f = F_a + F_s$ = 5988.11 kN

  Assuming full shear connection,

  $$\text{No. of shear connectors} = \frac{5988.11}{85} = 70$$

  $$\text{Spacing} = \frac{5485.4}{70} = 77\text{ mm}$$

  Let us provide 22mm diameter shear studs @55mm c/c throughout the span

9. <u>TRANSVERSE REINFORCEMENT</u>

Assuming 0.2 % reinforcement (perpendicular to the beam) for the solid slab,

Ac = 0.2/100 x 125 x 1000 = 250mm$^2$

Provide 8mm diameter @200mm c/c

OTHER BEAMS

SHORTER DIRECTION

Beam 2

This beam is for the third ,fourth and fifth floor .

ISMB 450 @ 0.724 KN/m

Maximum Bending Moment ($M_{u(+ve)}$) = 1449.789 KN m

Maximum Bending Moment ($M_{u(-ve)}$) = 821.236 KN m

Maximum Shear Force = 507.435 KN

LONGER DIRECTION

Beam 3

This beam is used for the first and the second floor.

ISMB 400@ 0.442 KN/m

Maximum Bending Moment ($M_{u(+ve)}$) = 426.541 KN m

Maximum Bending Moment ($M_{u(-ve)}$) = 418.718 KN m

Maximum Shear Force = 192.198 KN

Beam 4

This beam is used for the third, fourth and fifth floor.

ISMB 250 @ 0.373 KN/m

Maximum Bending Moment ($M_{u(+ve)}$) $=$ 255.141 KN m
Maximum Bending Moment ($M_{u(-ve)}$) $=$ 248.131KN m
Maximum Shear Force $=$ 111.594 KN

## 5.3 DESIGN OF COLUMN

The columns are designed as per EC4.The column design shown below is the design of the column for the first and second floors. The Axial load and Bending moments were obtained from STAAD analysis.

Design Axial Load, $F_x$ = 6201.959 kN.m.
Design Bending Moment about x-x axis, $M_{ux}$ = 1359.377 kN.m.
Design Bending Moment about y-y axis, $M_{uy}$ = 416.74 kN.m.

Considering 50% of the load,

$$F_x = 0.66 \times f_y \times A_a$$

Therefore, $$A_a = \frac{6201.95 \times 10^3}{2 \times 0.66 \times 250}$$

$$= 18793.2 mm^2$$

Try ISHB 450@ 0.925kN/m with additional plates on both flanges. The thickness of each plate is 25mm.

DETAILS OF THE SECTION

Column dimension = 600 x 600 x 3660

Concrete Grade = M 25

Steel Section = ISHB 450 with 2 plates of thickness 25mm on both the flanges

Steel Reinforcement = Fe 415

4 Nos. of 25mm dia. Bars

## DESIGN CALCULATIONS

### (1)MATERIAL PROPERTIES

1. Structural Steel

   Steel Section ISHB 450 with 2 plates of th. - 25mm on both the flanges

   Nominal Yield Strength, $f_y = 250$ N/mm$^2$

   Modulus of Elasticity, $E_a = 200$ k N/mm$^2$

2. Concrete

   Concrete Grade = M25

   Characteristic strength,$(f_{ck})_{cu} = 25$ N/mm$^2$

   Secant M .O.E. for short term loading, $E_{cm} = 31220$ N/mm$^2$

3. Reinforcing Steel

   Steel Grade = Fe 415

   Characteristic strength ,$f_{sk} = 415$ N/mm$^2$

   Modulus of Elasticity ,$E_s = 200$ k N/mm$^2$

4. Partial Safety Factors

   $\gamma_a = 1.15$

   $\gamma_c = 1.5$

   $\gamma_s = 1.15$

### (2)SECTION PROPERTIES

1. Steel Section

   $A_a = 27789$ mm$^2$

   $t_f = 13.7$ mm

   $t_p = 25$ mm

$h = 500$ mm

$t_w = 11.3$ mm

$I_{ax} = 1307.35 \times 10^6 \text{ mm}^4$

$I_{ay} = 166.91 \times 10^6 \text{ mm}^4$

$Z_{px} = 5798.8 \times 10^3 \text{ mm}^3$

$Z_{px} = 1721.62 \times 10^3 \text{ mm}^3$

2. Reinforcing Steel

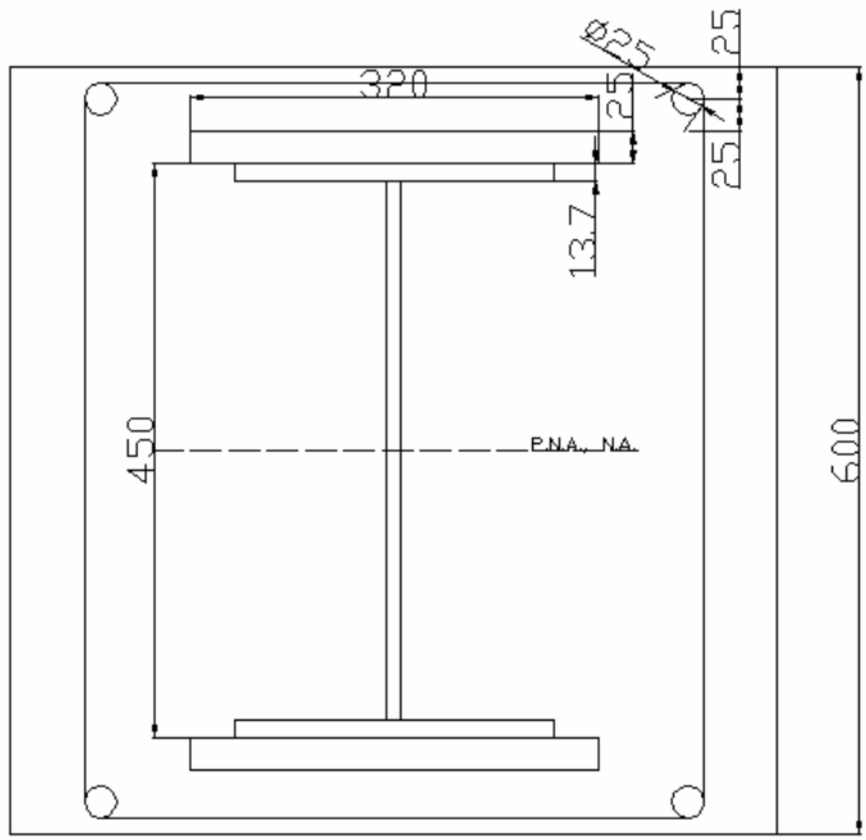
   4 bars of 25 mm dia. $A_s = 1963.5 \text{ mm}^2$

3. Concrete

   $A_c = A_{gross} - A_a - A_s$

   $= 600 \times 600 - 27789 - 1963.5$

   $= 330.25 \times 10^3 \text{mm}^2$



(3)DESIGN CHECKS

i. Plastic resistance of the section

$$P_p = \frac{A_a f_y}{\gamma_a} + \frac{\alpha_c A_c \left(f_{ck}\right)_{cu}}{\gamma_c} + \frac{A_s f_{sk}}{\gamma_s}$$

$$= \frac{27789 \times 250}{1.15} + \frac{0.85 \times 20 \times 27789}{1.5} + \frac{1963.5 \times 415}{1.15}$$

= **7064.59 kN**

ii. Effective elastic flexural stiffness of the section for short term loading

About the major axis

$$\left(EI\right)_{ex} = E_a I_{ax} + 0.8 E_{cd} I_{cx} + E_s I_{sx}$$

$$E_{cd} = \frac{31220}{1.35} = 23125 N / mm^2$$

$$I_{sx} = A\, h^2$$

$$= 1963.5 \times \left(\frac{600}{2} - 25 - \frac{25}{2}\right)^2$$

$= 135.3 \times 10^6\ mm^4$

$$I_{cx} = \frac{600^4}{12} - \left(135.3 + 1307.35\right) \times 10^6$$

$= 9357.36 \times 10^6\ mm^4$

$(EI)_{ex} = 2 \times 10^5 \times 1307.35 \times 10^6 + 0.8 \times 23125 \times 9357.36 \times 10^6$

$+ 2 \times 10^5 \times 135.3 \times 10^6$

$= 461.64 \times 10^{12}\ N.mm^2$

About the minor axis

$$\left(EI\right)_{ey} = E_a I_{ay} + 0.8 E_{cd} I_{cy} + E_s I_{sy}$$

$$I_{sy} = A h^2$$

$$= 1963.5 \times \left( \frac{600}{2} - 70 - \frac{25}{2} \right)^2$$

$= 92.88 \times 10^6\ mm^4$

$I_{cy} = 600^4/12 - (92.88 + 166.9) \times 10^6$

$= 10540.24 \times 10^6\ mm^4$

$(EI)_{ey} = 2 \times 10^5 \times 166.91 \times 10^6 + 0.8 \times 23125 \times 10540.24 \times 10^6$

$+ 2 \times 10^5 \times 92.88 \times 10^6$

$= 246.95 \times 10^{12}\ N.mm^2$

iii. Non dimensional slenderness

$$\lambda = \left( \frac{P_{pu}}{P_{cr}} \right)^{1/2}$$

Value of $P_{pu}$

$P_{pu} = A_a f_y + \alpha_c A_c (f_{ck})_{cy} + A_s f_{sk}$

$P_{pu} = 27789 \times 250 + 0.85 \times 330.25 \times 10^3 \times 20 + 415 \times 1963.5$

$=$ **13374.342kN**

$$(P_{cr})_x = \frac{\pi^2 (EI)_{ex}}{l^2}$$

$$= \frac{\pi^2 \times 461.632 \times 10^{12}}{3660^2}$$

$= 340121\ kN$

$$\lambda_x = \left( \frac{13374.324}{340121} \right)^{1/2}$$

$=$ **0.21**

$$(P_{cr})_y = \frac{\pi^2 (EI)_{ey}}{l^2}$$

$$= \frac{\pi^2 \times 246.95 \times 10^{12}}{3660^2}$$

$=181947\ kN$

$$\lambda_y = \left(\frac{13374.324}{1819471}\right)^{1/2}$$

$= \underline{\mathbf{0.27}}$

iv. Check for long term loading can be neglected if anyone or both following conditions are satisfied

- Eccentricity, e given by

  e = $\frac{M}{F_x} \geq 2$ times the cross section dimension in the plane of bending considered

  $$e_x = \frac{1359.377}{6201.959} = 0.22 < (2 \times 0.6)$$

  $$e_y = \frac{416.742}{6201.959} = 0.06 < (2 \times 0.6)$$

- $\lambda < 0.8$

Since both the conditions are satisfied, the influence of creep and shrinkage on the ultimate load need not be considered.

v. Resistance of the composite column under axial compression

Design against axial compression is satisfied if the following condition is satisfied:

$F_x < \chi P_p$

$\chi$ = reduction factor for column buckling

$\chi$ values:

About major axis

$\alpha_x = 0.34$

$\varphi_x = 0.5\ (1 + \alpha_x\ (\lambda_x - 0.2) + \lambda_x^{\ 2})$

$= 0.5\ (1 + 0.34\ (0.21 - 0.2) + 0.21^2$

$= 0.52$

$$\chi_x = \frac{1}{\left\{\varphi + \left(\varphi^2 - \lambda_x^{\ 2}\right)^{1/2}\right\}}$$

$$= \frac{1}{\left\{0.52 + \left(0.52^2 - 0.21^2\right)^{1/2}\right\}}$$

$= 0.99$

About minor axis

$\alpha_y = 0.49$

$\varphi_y = 0.5\ (1 + \alpha_y\ (\lambda_y - 0.2) + \lambda_y^{\ 2}$

$= 0.5\ (1 + 0.49\ (0.27 - 0.2) + 0.27^2$

$= 0.55$

$$\chi_y = \frac{1}{\left\{\varphi + \left(\varphi^2 - \lambda_y^{\ 2}\right)^{1/2}\right\}}$$

$$= \frac{1}{\left\{0.55 + \left(0.55^2 - 0.27^2\right)^{1/2}\right\}}$$

$= 0.97$

$\chi_x\ P_{px} = 0.99 \text{ x } 7064.59 =$ **6993.9kN>F$_x$**

$\chi_y\ P_{py} = 0.97 \text{ x } 7064.59 =$ **6836.9kN> F$_y$**

Therefore the design is O.K. for axial compression.

vi. Check for second order effects

Isolated non-sway columns need not be checked for second order effects if

i. $\frac{F_x}{(P_{cr})_x} \leq\ 0.1$ for major axis bending

$$\frac{6201\ .959}{340121} =\ 0.01 < 01$$

ii. $\frac{F_x}{(P_{cr})_y} \leq\ 0.1$ for minor axis bending

$$\frac{6201\ .959}{1181947} =\ 0.03 < 01$$

Therefore, check for second order effects is not necessary.

vii. <u>Resistance of the composite column under uniaxial compression and biaxial bending</u>

Compressive resistance of concrete, $Pc = A_c p_{ck}$

$$= 330.25 \text{ x } 10^3 \text{ x } 14.17$$

$$= 4679.5 \text{ kN}$$

<u>About major axis</u>

Plastic section modulus of the reinforcement

$$Z_{ps} = 4\left(\frac{\pi \times 25^2}{4}\right)\left(\frac{600}{2} - 25 - \frac{25}{2}\right)$$

$$= 515.42 \text{ x } 10^3 \text{ mm}^3$$

Plastic section modulus of the steel section,

$Z_{pa} = 5798.8 \text{ x } 10^3 \text{ mm}^3$

Plastic section modulus of the concrete,

$$Z_{pc} = \frac{b_c h_c^{\ 2}}{4} - Z_{ps} - Z_{pa}$$

$$= \frac{600^3}{4} - 515.42 \times 10^3 - 5798.8 \times 10^3$$

$$= 47685.78 \text{ x } 10^3 \text{ mm}^3$$

Check that the position of the neutral axis is in the web,

$$h_n = \frac{A_c p_{ck} - A_s^{'}(2p_{sk} - p_{ck})}{2b_c p_{ck} + 2t_w(2p_y - p_{ck})}$$

$$= \frac{(330.25 \times 10^3 \times 14.17)}{2 \times 600 \times 14.17 + 2 \times 11.3\left(2\frac{250}{1.15} - 14.17\right)}$$

$$= 176.52 \text{ mm} < (0.5h - t_f)$$

The neutral axis is in the web.

($A'_s = 0$ as there is no reinforcement within the region of the steel web)

Section modulus about neutral axis,

$Z_{psn}$ = 0 (as there is no reinforcement within the region of the steel web)

$Z_{pan} = t_w h_n^2$

$= 11.3 \text{ x } 176.52^2$

$= 352100.2 \text{ mm}^3$

$Z_{pcn} = b_c h_n^2 - Z_{psn} - Z_{pan}$

$= 600 \text{ x } 176.52^2 - 352100.2$

$= 18343.49 \text{ x } 10^3 \text{ mm}^3$

Plastic moment of resistance of section,

$M_{px} = p_y (Z_{pc} - Z_{pan}) + 0.5p_{ck} (Z_{pc} - Z_{pcn}) + p_{sk} (Z_{ps} - Z_{psn})$

$= 217.4 \text{ ( } 5798.8 \text{ x } 10^2 - 352100.2 \text{ )}$

$+ 0.5 \text{ x } 14.17 \text{ x } (47685.78 \text{ x } 10^3 - 18343.49 \text{ x } 10^3)$

$+ 361 \text{ x } (515.42 \text{ x } 10^3)$

= **<u>12924.00 kN.m</u>**

<u>About minor axis</u>

Plastic section modulus of the reinforcement,

$$Z_{ps} = 4\left(\frac{\pi \times 25^2}{4}\right)\left(\frac{600}{2} - 70 - \frac{25}{2}\right)$$

$= 427.06 \text{ x } 10^3 \text{ mm}^3$

Plastic section modulus of the steel section,

$Z_{pa} = 1721.62 \text{ x } 10^3 \text{ mm}^3$

Plastic section modulus of the concrete,

$$Z_{pc} = \frac{b_c h_c^2}{4} - Z_{ps} - Z_{pa}$$

$$= \frac{600^3}{4} - 427.06 \times 10^3 - 1721.62 \times 10^3$$

$= 51851.32 \text{ x } 10^3 \text{ mm}^3$

$$h_n = \frac{A_c p_{ck} - A_s'(2p_{sk} - p_{ck}) + t_w(2t_f - h)(2p_y - p_{ck})}{2h_c p_{ck} + 4t_f(2p_y - p_{ck})}$$

$$= \frac{330.25\times10^3\times14.17+11.3(2\times13.7-500)(2\times217.4-14.17)}{2\times600\times14.17+413.7(2217.4-14.17)}$$

$= 60.8$ mm $< (0.5t_w < h_n < 0.5b$

The neutral axis is in the web.

($A'_s = 0$ as there is no reinforcement within the region of the steel web)

Section modulus about neutral axis,

$Z_{psn} = 0$ (as there is no reinforcement within the region of the steel web)

$Z_{pan} = 2t_f h_n^2 + (h - 2t_f) /4 \text{ x } t_w^2$

$= 2 \text{ x } 13.7 \text{x } 60.8^2 + (500 - 2 \text{ x } 13.7)/4 \text{ x } 11.3^2$

$= 116.37 \text{ x } 10^3 \text{ mm}^3$

$Z_{pcn} = h_c h_n^2 - Z_{psn} - Z_{pan}$

$= 550 \text{ x } 60.8^2 - 116.37 \text{ x } 10^3 \text{ mm}^3$

$M_{py} = p_y (Z_{pc} - Z_{pan}) + 0.5p_{ck} (Z_{pc} - Z_{pcn}) + p_{sk} (Z_{ps} - Z_{psn})$

$= 217.4 \text{ x } (1721.62 \text{ x } 10^3 - 116.37 \text{ x } 10^3)$

$+ 0.5 \; 14.17 \; ( 51851.3 \text{ x } 10^3 - 1916.78 \text{ x } 10^3 )$

$+ 361 \text{ x } 427.06 \text{ x } 10^3$

= **<u>856.94 kN.m</u>**

viii. <u>Check of column resistance against combined compression and bi-axial bending</u>

The design against combined compression and bi-axial bending is adequate if the following conditions are satisfied.

(1) $M < 0.9 \, \mu \, M_p$

About major axis

$M_{ux} = 1359.377$ kN.m.

$\mu_x$ = moment resistance ratio $= 1 - \left\{ \frac{(1-\chi_x)\chi_d}{(1-\chi_c)\chi_x} \right\}$

$$= 1 - \left\{ \frac{(1 - 0.99)0.88}{(1 - 0.66)0.99} \right\}$$

$$= 0.97$$

$M_{ux} < 0.9\ \mu_x\ M_{px}$

$< 0.9 \times 0.97 \times 12924.50 = 11327.51$ kN.m

About minor axis

$M_{uy} = 416.74$ kN.m

$\mu_y$ = moment resistance ratio $= 1 - \left\{ \frac{(1 - \chi_y)\chi_d}{(1 - \chi_c)\chi_y} \right\}$

$$= 1 - \left\{ \frac{(1 - 0.97)0.88}{(1 - 0.66)0.97} \right\}$$

$$= 0.92$$

$M_{uy} < 0.9\ \mu_x\ M_{px}$

$< 0.9 \times 0.92 \times 856.94 = 709.55$ kN.m

$$(2) \frac{M_{ux}}{\mu_x M_{px}} + \frac{M_{uy}}{\mu_y M_{py}} < 1.0$$

$$\frac{1359.377}{0.97 \times 12924} + \frac{416.742}{0.92 \times 704.55} = 0.74 < 1$$

Therefore , the design is safe.

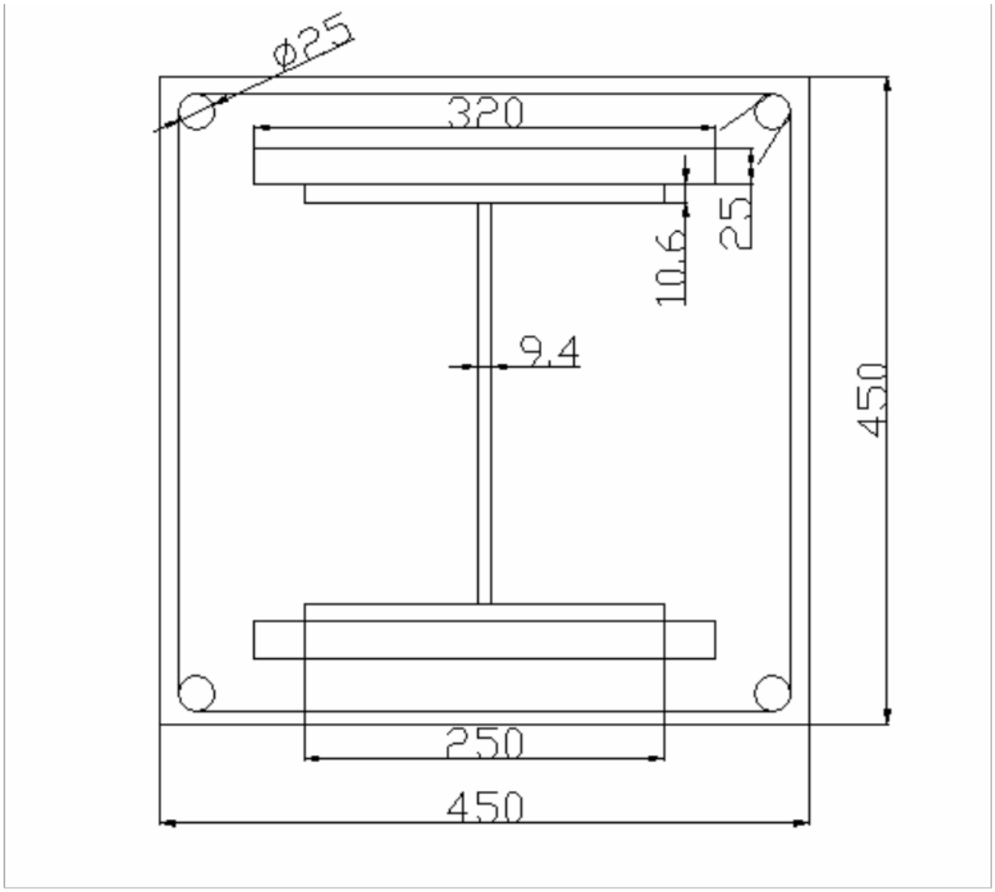
OTHER COLUMNS

Column 2

This column cross-section used for the third ,fourth and fifth floor.

ISHB 300@0.63kN/m (with 25mm thick plates on the top and bottom of both the flanges. Both the plates are of 320mm width

$M_{ux}$ = 787.405 KN m
$M_{uy}$ = 292.717 KN m
$F_x$ = 2618.69 KN




## 5.4 DESIGN OF FOUNDATION

The foundation is designed as a square footing.

The load safe bearing capacity of the soil is assumed as 250 kN / m$^2$

1. PLAN DIMENSIONS OF THE FOOTING

   Factored load, $P_u$ = 6201.959 kN.m

   Service load, $P_s \quad = \frac{6201.959}{1.5}$ = 4134.4 kN.m

   Take weight of the foundation @10% = 413.4 kN.m

   Therefore total load = 4134.4 + 413.4 = 4547.7 kN.m

   Safe bearing capacity = 250 kN / m$^2$

Area required = $\frac{Load}{S.B.C.} = \frac{4577.7}{250.} = 18.19\ mm^2$

Side of the footing = L = 4.2 m

2. STRUCTURAL DESIGN OF FOUNDATION

Net Upward Pressure ,$q_u = \frac{P_u}{L^2}$

$= 6201.959 / 4.2^2$

$= 351.59\ kN / m^2$

3. DESIGN FOR BEAM SHEAR

Assume minimum percentage of steel provided in the footing slab as

0.12%

From Table 19 of IS:456-2000

$\tau_c = 0.28\ N/mm^2$

Let d is the effective depth required

Let L be the size of the footing

Let a be the side of the column

Let $V_u$ be the factored S.F. in the footing at a critical section of d m from the face of the column

$$d = \frac{q_u(L-a)}{2(q_u+\tau_c)}$$

$$= \frac{351.59\times10^{-3}(4.2-0.6)}{2(351.59\times10^{-3}+0.28)}$$

$= 1.798$ m

Assuming a clear cover of 50 mm and 20mm dia.bars

D = 1.798 + 0.05 + 0.02/2 = 2.308 m $\approx$ 2.40 m

d = 2400 – 50 – 20 /2 = 2340 mm

4. CHECK FOR PUNCHING SHEAR

Let $V_{u,p}$ be the factored Punching shear

$$V_{u,p} = q_u\left(L^2-(a+d)^2\right)$$

$= 351.59 \times 10^{-3} \ (4200^2 - (600 + 2340)^2$

$= 3613$ kN

Permissible stress , $\tau_{c,p} = 0.25\sqrt{f_{ck}} = 0.25\sqrt{25}$

$= 1.25$

$$\tau_{c,p} = \frac{V_{up}}{4(a+d)d}$$

$$= \frac{3613 \times 1000}{4(600+2340)2340}$$

$= 0.11 < 1.25$

5. CHECK FOR BENDING MOMENT

$$M_u = \frac{P_u(L-a)^2}{8L}$$

$$= \frac{6201.959(4.2-0.6)^2}{8 \times 4.2}$$

$= 2392.18$ kN.m

$M_{u,lim} = 0.36 \times 0.48 \ (1 - 0.42 \times 0.40) \times 25 \times 4200 \times 2340^2$

$= 79320.5$ kN.m $> M_u$

$$Ast = \frac{2340 \times 4200 \times 25}{2 \times 415}\left(1 - \sqrt{1 - \frac{4.598 \times 2392.2 \times 10^6}{4200 \times 2340^2 \times 25}}\right)$$

$= 2845 \text{ mm}^2$

These bars are provided for the full width of the footing i.e. 4.2 m

Using 20 mm diameter bars,

no. of bars $= \dfrac{2845}{\dfrac{\pi \times 25^2}{4}} = 6$

Provide these bars in both the directions

# CHAPTER 6.

# DESIGN OF ELEMENTS (R.C.C.)

## 6.1 DESIGN OF BEAMS

The beams are designed using SP – 16

BEAM 1-LONGER SPAN

Maximum positive B.M $M_u$(+ve)= 1617.087 kN.m

Maximum negative B.M $M_u$(-ve)= 980.359 kN.m

Maximum S.F $V_u$= 547.085 kN.

1. Initially a beam of size 300 x 900 is selected

   Here,b = 300mm

   d = 850 mm

2. $\frac{M_u}{bd^2} = \frac{1617.087\times10^6}{300\times850^2} = 7.46$

   $p_t = 2.44\%$
   $p_c = 1.32\%$

   Steel in tension $= \frac{2.44\times300\times850}{100} = 6222mm^2$

   Steel in compression $= \frac{6222\times1.32}{2.44} = 3366mm^2$

   Considering 60% of steel,

   Steel in compression $= 3366\times{}^{60}\!/_{100} = 2020mm^2$

   Total amount of steel $= 6222 + 2020 = 8242mm^2$

   Total volume of steel $= 8242\times9412 = 77.57\times10^6\,mm^3$

   Total weight of steel $= 77.57\times10^{-3}\times7800 = 605.046kg$

3. Providing steel reinforcement of 8mm diameter bars @200mmc/c

Steel Reinforcement $=\dfrac{2400\times9142\times50\times7800\times10^{-9}}{200}=42.78kg$

4. Total steel $=605.046+42.78\approx650kg$

5. Total volume of concrete $=9.142\times0.3\times0.9=2.46m^{3}$

<u>BEAM 2 –SHORTER SPAN</u>

Maximum positive B.M $M_u$(+ve)= 426.541 kN.m

Maximum negative B.M $M_u$(-ve)= 418.718 kN.m

Maximum S.F ,$V_u$= 192.198 kN.

1. Initially a beam of size 250 x 600 is selected

   Here,b = 250mm

   d =550mm

2. $\dfrac{M_u}{bd^2}=\dfrac{426.541\times10^{6}}{250\times550^{2}}=5.6$

   $p_t=1.856\%$
   
   $p_c=0.7\%$

   Steel in tension $=\dfrac{1.856\times250\times550}{100}=2565.75mm^{2}$

   Steel in compression $=\dfrac{2565.75\times0.7}{1.856}=967.69mm^{2}$

   Considering 60% of steel,

   Steel in compression $=967.69\times{}^{60}\!/_{100}=580.614mm^{2}$

   Total amount of steel $=2565.75+580.614=3146.36mm^{2}$

   Total volume of steel $=3146.36\times4500=14.158\times10^{6}\,mm^{3}$

   Total weight of steel $=14.158\times1000\times7800\times10^{-9}=110.44kg$

3. Providing steel reinforcement of 8mm diameter bars @200mmc/c

   Steel Reinforcement $=\dfrac{2400\times4500\times50\times7800\times10^{-9}}{200}=21.06kg$

4. Total steel $=110.44+21.06=131.5kg=131.5kg$

5. Total volume of concrete $=4.5\times0.25\times0.6=0.675m^{3}$

## 6.2 DESIGN OF COLUMN

The design of the column was carried out as per SP-16

Design Axial Load, $F_x$ = 6201.959 kN.m.

Design Bending Moment about x-x axis, $M_{ux}$ = 1359.377 kN.m.

Design Bending Moment about y-y axis, $M_{uy}$ = 416.74 kN.m.

1. Resultant Moment $= 1.15\sqrt{\left(1359.38^2 + 416.742^2\right)} = 1635.096 kN.m$
2. Considering a section of 500 x 900 mm,
3. Using 4% of reinforcement,

$$\frac{P_u}{f_{ck}bd} = \frac{6201.96\times1000}{25\times900\times500} = 0.55$$

$$\frac{M_u}{f_{ck}bd^2} = \frac{1635.096\times10^6}{25\times900^2\times500} = 0.16$$

$$\frac{p}{f_{ck}} = 0.22$$

Therefore p = 5.5%

4. Weight of steel $= \frac{5.5}{100}\times900\times500\times3.66\times1000\times7800\times10^{-9}$

$= 173.74 kg \approx 200 kg$

5. Volume of concrete $= 0.9\times0.5\times3.66 = 1.647 m^3$

## 6.3 DESIGN OF FORMWORK

1. **For Short beam**

   Consider 1m length,

   Formwork required $= (0.3+0.9+0.9)\times1 = 2.1 m^2/m = 19.194 sq.m$

2. **For Long beam**

   Formwork required $= (0.25+0.6+0.6)\times1 = 1.45 m^2/m = 6.525 sq.m$

3. **For Column**

   Formwork required $= (0.5+0.5+0.9+0.9)\times1 = 2.8 m^2/m$

# CHAPTER 7

# COMPARISON OF COMPOSITE ELEMENTS WITH CONVENTIONAL RCC ELEMENTS

## 7.a SLABS

Consider 1m length of the slab,

| Material | Rate | Composite design | Amount | R.C.C.design | Amount |
|---|---|---|---|---|---|
| Steel | Rs.35/kg | 9.36kg/sq.m | Rs.328 | 3.9kg/m | Rs.136.5 |
| Concrete | Rs.2050/$m^3$ | 0.075 $m^3$/m | Rs.153.75 | 0.15 $m^3$/m | Rs.307.5 |
| Formwork | Rs.80/sq.m | - | - | 0.075sq.m/m | Rs.6 |
| | | Total | Rs.481.75 | Total | Rs.447 |

## 7.b BEAMS

a. Considering shorter span beams,

| Material | Rate | Composite design | Amount | R.C.C.design | Amount |
|---|---|---|---|---|---|
| Steel | Rs.35/kg | 1839kg | Rs.64365 | 650kg | Rs.22750 |
| Concrete | Rs.2050/$m^3$ | - | - | 2.46 $m^3$ | Rs.5060 |
| Formwork | Rs.80/sq.m | - | - | 19.194sq.m | Rs.1535.5 |
| | | Total | Rs.64365 | Total | Rs.24792 |

**7.c**

b. Considering longer span beams,

| Material | Rate | Composite design | Amount | R.C.C.design | Amount |
|---|---|---|---|---|---|
| Steel | Rs.35/kg | 277.2kg | Rs.9702 | 131.5kg | Rs.4602.5 |
| Concrete | Rs.2050/$m^3$ | - | - | 0.675 $m^3$ | Rs.1383.75 |
| Formwork | Rs.80/sq.m | - | - | 6.525sq.m | Rs.522 |
| | | Total | Rs.9702 | Total | Rs.6508.25 |

**7.d COLUMNS**

| Material | Rate | Composite design | Amount | R.C.C.design | Amount |
|---|---|---|---|---|---|
| Steel | Rs.35/kg | 798.3.9kg | Rs.27939 | 200kg | Rs.7000 |
| Concrete | Rs.2050/$m^3$ | 1.21 $m^3$ | Rs.2480.5 | 1.647 $m^3$ | Rs.3376.35 |
| Formwork | Rs.80/sq.m | 2.4 sq.m | Rs.192 | 10.248sq.m | Rs.820 |
| | | Total | Rs.30611 | Total | Rs.11197 |

# CHAPTER 8

## SUMMARY AND CONCLUSION

1) A G + 5 structure of plan dimensions 56.3m x 31.94m has been analysed, designed and cost per unit quantities worked out.
2) An equivalent R.C.C. structure has also been analyzed, designed and cost per unit quantities worked out.
3) (A) A comparative study of the quantity of material and cost has been worked out both for composite and concrete construction.

(B) Though, the cost comparison reveals hat Steel-Concrete composite design structure is more costly, reduction in direct costs of steel-composite structure resulting from speedy erection will make Steel-Composite structure economically viable. Further, under earthquake considerations because of the inherent ductility characteristics, Steel-Concrete structure will perform better than a conventional R.C.C. structure.

4) For analysis,STAADPro-2003 software has been used.
5) Manual design has been carried out both for Steel-Concrete composite and R.C.C. structure.
6) Sufficient insight into the analysis and design of Steel-Concrete composite structure which is an emerging area has been gained
7) Immense confidence has been gained in the analysis and design of a multi-storeyed structure using STAAD Pro 2003 software which will benefit us as we step out of the portals of the college.

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