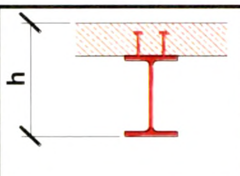
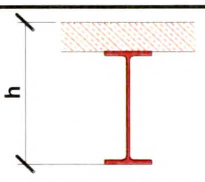
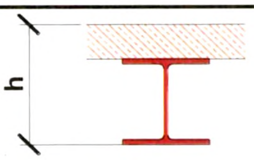
13. Parametric Study of a Composite Building

13.1 General remarks

As mentioned earlier, composite frames commonly used for buildings usually comprise a bare-steel frame of H-section columns supporting I-section beams, laid out in a rectangular grid of primary (shortest span) and secondary members, supporting an overlaid composite floor deck. The composite flooring deck system consists of cold-formed profiled steel sheets which act not only as the permanent formwork for an in situ cast concrete slab but also to some extent as tensile reinforcement. Even combination of concrete cores, steel frame and composite floor construction is the standard system for tall buildings as they are best suited to resist repeated earthquake loadings, which require a high amount of resistance and ductility. Now it is rapidly gaining the status as the most preferred type of construction by many architects, engineers, and developers.

Table 13.1 Composite Beam versus Steel Beam

Beam Parameters			
	Composite beam	Steel beam without any shear connection	
Steel cross section	IPE 400	IPE 550	HE 360 B
Construction height [mm]	560	710	520
Load capacity	100%	100%	100%
Steel weight	100%	159%	214%
Construction height	100%	127%	93%
Stiffness	100%	72%	46%

The use of composite action has certain advantages. In particular, a composite beam has greater stiffness and usually a higher load resistance than its non-composite counterpart. A number of parameters for composite beam are compared in **Table 13.1** with two types of steel beams having I- and H- cross sections with no shear connection to the concrete slab.

The load capacity is nearly the same but the difference in stiffness and construction height is noticeable. Consequently, a smaller steel section is usually required. The result is a saving of material and depth of construction. In turn, the latter leads to lower storey heights in buildings.

The use of limit state methods to determine strength is common when dealing with composite construction. Recently, IS: 800-2007 code for steel structure has also been upgraded as per limit state method. Analysis and design of some simple non-sway frames under gravity load can be carried out manually. But, for the detailed analysis of large frame, with high indeterminacy, a large number of computer software packages (such as STAAD.Pro V8i, ANSYS, NISA, SAP etc) are available in the market and are increasingly being used by designers worldwide.

Although composite steel-concrete structures are economic construction but verifications that are required for analysis and design are tedious. This had led numerous researchers to develop methods so that engineer can do design immediately and verify the answer with number of alternatives. Here a parametric study of composite steel-concrete structure and its analysis and design having different component combinations like composite slabs with different profiled sheets is carried out utilizing the software STAAD Pro V8i Modelling covers input of building geometry and loading condition. Using different types of beam sections, different types of loads, various country codes, orientation of column and a comparison with light weight concrete to the conventional concrete are some of the important aspects included in the parametric study.

13.2 MOMENT-ROTATION CURVES

Although there are numerous composite beam options available for use in simple construction, connections have not yet been developed that would allow many of these to be incorporated in a continuous (or semi-continuous) composite frame. The importance of the detailing on the connection performance can be appreciated by referring to **Fig. 13.1**. This figure shows typical (simplified bi-linear) moment-rotation responses for steel and composite connections. In addition to a response for a pinned connection, one curve indicates the performance that could be expected from a standard flush end plate bare steel connection, and the second curve indicates the performance that could be expected from a composite connection achieved by simply adding some slab reinforcement in the connection zone (whilst maintaining the same steelwork detailing). A line of constant moment on the figure indicates the moment resistance of the beam itself. The following information can be identified from the figure: It can readily be seen that the composite connection is both stiffer and stronger than the bare steel connection. This can only be achieved when sufficient reinforcement can be located and anchored in the slab. The use of a steel deck based slab with in-situ concrete facilitates the incorporation of this reinforcement. Both bare steel and composite connections may be what is known as partial strength. This means that they have a lower moment resistance than that of the adjacent beam [2].

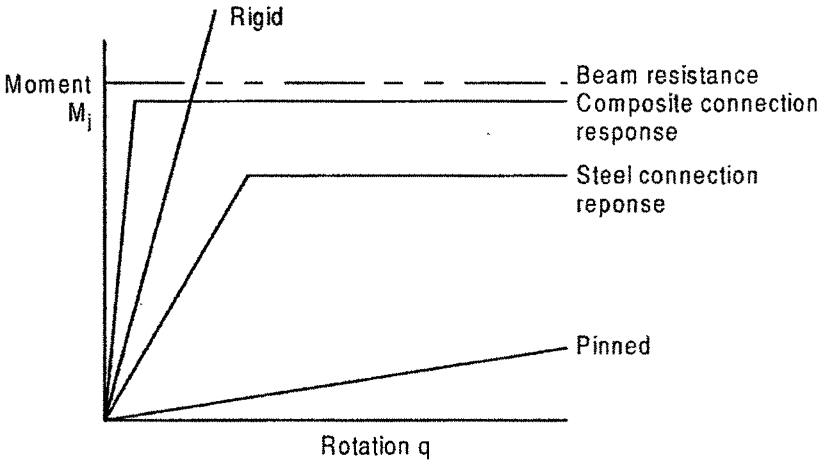


Fig. 13.1 Bi-Linear Moment-Rotation Curves for Steel and Composite Connections

The use of connections that provide any reasonable degree of continuity, expressed in terms of the partial strength of the connection, can be beneficial. The stiffness of the connections is also beneficial in reducing beam deflections considerably. Clearly, however, the use of composite connections will allow more substantial reductions in sagging moments than when weaker bare steel connections are used. One of the great benefits of semi continuous construction is that the beams and connections can be 'balanced'; savings in beam depth or weight can be weighed-up against connection costs to achieve an optimum solution.

13.3 Types of Frames

13.3.1 Braced Frames

Braced frames may be grouped into concentrically braced frames (CBFs), and eccentrically braced frames (EBFs), depending on their geometric characteristics. In CBFs, the axes of all members i.e., columns, beams, and braces-intersect at a point such that the member forces are axial as shown in **Fig. 13.2**. EBFs utilize axis offsets to deliberately introduce flexure and shear into framing beams as shown in **Fig. 13.3**. The primary goal is to increase ductility. In addition, concentrically braced frames are subdivided into two categories, namely, ordinary concentrically braced frames (OCBFs) and special concentrically braced frames (SCBFs).

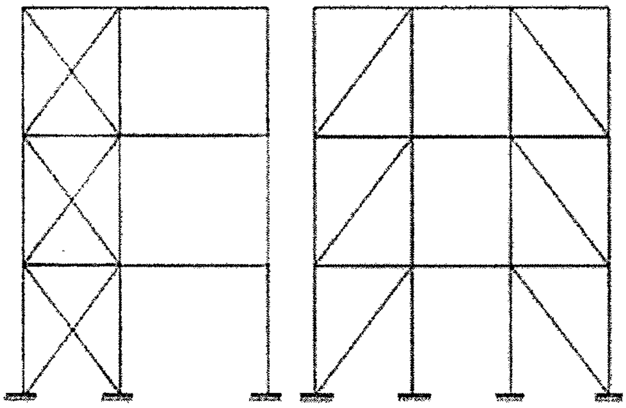


Fig. 13.2 Concentric Truss Bracings

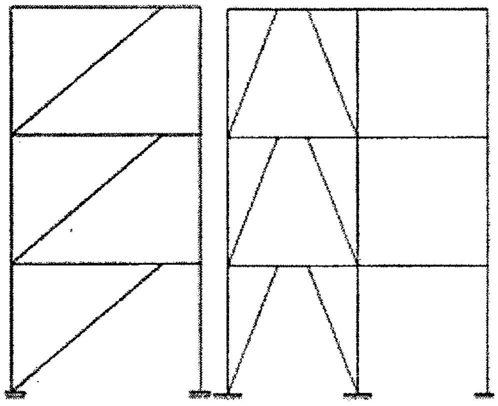


Fig. 13.3 Eccentric Truss Bracings

13.3.2 UNBRACED FRAMES

In frames where lateral stability depends upon the bending stiffness of rigidly connected beams and columns, the effective length factor K of compression members shall be determined by structural analysis. Stiffness reduction adjustments due to column inelasticity are permitted. Analysis of the required strength of unbraced multistory frames shall include the effect of frame instability and column axial deformation under factored load combination.

13.3.3 SWAY-NON SWAY FRAMES

A 'non-sway' frame is one for which the sway deflections are sufficiently small to make second order forces and moments negligible. Some configurations of bracing allow significant sway, so that the frame analysis must then consider second order effects; either explicitly or by using a simplification such as the amplified sway method [93]. When the global second-order effects are not negligible, the frame is said to be sway frame. Alternatively, the design of the columns may allow for the second order effects by using effective lengths in excess of the system lengths.

In non-sway (braced) frames, the column buckle in single curvature and hence their effective length factor will always be less that unity; whereas the columns in sway frames buckle in double curvature and hence, their effective length factor will always be greater than unity. Also, the condition of sway stability imposes that there must not be excessive lateral deformation under applied loads.

13.4 LINEAR STATIC ANALYSIS

The 'lateral force' method is a simplified version of the modal response method and is a static analysis which can only be employed for regular structures which respond essentially in single mode of vibration. This approach defines a series of forces acting on a building to represent the effect of earthquake motion, typically defined by a seismic design response spectrum. It assumes that the building responds in its fundamental mode. For this to be true, the building must be low-rise and must not twist significantly when the ground moves. The response is read from a design response spectrum, given the natural frequency of the building. The applicability of this method is extended in many building codes by applying factors to account for higher buildings with some higher modes, and for low levels of twisting. To account for effects due to "yielding" of the structure, many codes apply modification factors that reduce the design forces. Similarly to the 'equivalent' force applied to the mass of the simple cantilever, it is possible to define in multi-storey buildings a set of 'storey' forces, which are applied at each storey level and which induce the same deformed shape as the earthquake.

As per IS 1893:2002 [114], following procedure should be followed to calculate earthquake generated forces and these forces should be applied at each storey level.

The total design lateral force or design seismic base shear (V_B) along any principal direction shall be determined by the following expressions:

$$V_B = A_h \times W \qquad \text{... (13.1)}$$

where, A_h = Design horizontal acceleration spectrum value and

W = Seismic weight of the building.

The design horizontal acceleration spectrum value for a structure shall be determined by the following expression:

$$A_h = \frac{ZIS_a}{2Rg} \qquad \text{... (13.2)}$$

Provided that for any structure with T < 0.1s, the value of A_h will not be taken less than Z/2 whatever be the value of I/R.

Here, Z = Zone factor given in **Table 13.2**, is for the Maximum Considered Earthquake (MCE) and service life of structure in a zone. The factor 2 in the denominator of Z is used so as to reduce the Maximum Considered Earthquake (MCE) zone factor to the factor for Design Basis Earthquake (DBE).

Table 13.2 Value of Zone Factor (Z)

Seismic Zone	II	III	IV	V
Seismic Intensity	Low	Moderate	Severe	Very Severe
Z	0.10	0.16	0.24	0.36

I = Importance factor **(Table 13.3)**, depending upon the functional use of the structures, characterized by hazardous consequences of its failure, post-earthquake functional needs, historical value or economic importance.

Table 13.3 Value of Importance Factor (I)

Sr. No.	Type of Structure	Importance Factor
1	Important service and community buildings, such as hospitals; schools; monumental structures; emergency buildings like telephone exchange, television stations, radio stations, railway stations; large community halls like cinemas, assembly halls and subway stations, power stations.	1.5
2	All other buildings	1.0

R = Response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. However, the ratio (I/R) shall not be greater than 1.0. The values of R for buildings are given in **Table 13.4**.

Table 13.4 Response Reduction Factor for Building Systems

Sr. No.	Building Frame Systems	R
1	Ordinary RC moment-resisting frame	3.0
2	Special RC moment-resisting frame	5.0
3	a) Steel frame with Concentric braces b) Steel frame with Eccentric braces	4.0 5.0
4	Steel moment resisting frame designed as per SP 6 (Part 6)	5.0

S_a / g = Average response acceleration coefficient

For rocky, or hard soil sites

$$
\begin{aligned}
S_a / g &= 1 + 15T && 0.00 \le T \le 0.10 \\
&= 2.50 && 0.10 \le T \le 0.40 \\
&= 1.00/T && 0.40 \le T \le 4.00
\end{aligned}
$$

For medium soil sites

$S_a / g \;= 1 + 15T \qquad 0.00 \leq T \leq 0.10$

$= 2.50 \qquad 0.10 \leq T \leq 0.55$

$= 1.36/T \qquad 0.55 \leq T \leq 4.00$

For soft soil sites

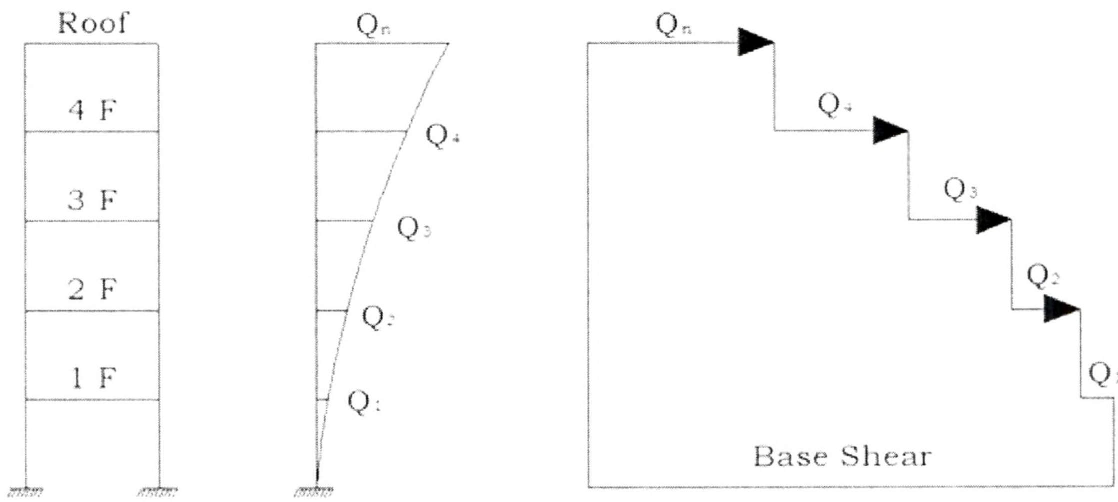
$S_a / g \;= 1 + 15T \qquad 0.00 \leq T \leq 0.10$

$= 2.50 \qquad 0.10 \leq T \leq 0.67$

$= 1.67/T \qquad 0.67 \leq T \leq 4.00$

Where, T = Time period (Time of oscillation) = $0.09h/(d)^{0.5}$ for structure with brick infill, h = Height of building in m, and d = Base dimension of the building at the plinth level in m, along the considered direction of the lateral force.

After finding the base shear V_B for the whole building, its distribution Q_i along each of the storey height of the building can be assumed in many ways. One type of distribution is used in the inverted triangle. However, IS 1893 assumes the parabolic distribution as indicated by the following equation and **Fig. 13.4**.



(a) Building (b) Distribution of Forces (c) Distribution of Base Shear

Fig. 13.4 Calculation of Base Shear

$$Q_i = V_B \frac{W_i {h_i}^2}{\sum_{i=1}^{n} W_i {h_i}^2} \qquad ...(13.3)$$

Where, Q_i = Design lateral force of floor i, W_i = Seismic weight of floor i, h_i = Height of floor 'i' measured from base, and n = Number of storeys.

For computation of W_i, the live load is to be 25% of live load if it is < 3 kN/m^2 and 50% if > 3 kN/m^2.

The force distribution in any storey is worked out from top as shown in **Fig. 13.3(c)**. It is given by the equations.

$V_n = Q_n$ at roof level

$V_i = V_{(i+1)} + Q_i$ at each floor level the corresponding force is added to get its shear force so that at the base the total shear calculated.

13.5 MODELLING OF BUILDINGS

Structural models are idealizations of the prototype and are intended to simulate the response characteristics of systems. These are summarized below in order of complexity and accuracy.

13.5.1 SUBSTITUTE (OR EQUIVALENT SDOF) MODELS

The structure is idealized as an equivalent single-degree of freedom (SDOF) system or 'substitute system'. Following four parameters are needed to define the substitute system: effective mass M_{eff}, effective height H_{eff}, effective stiffness K_{eff} and effective damping ξ_{eff}. The height H_{eff} defines the location of the equivalent or effective mass M_{eff} of the substitute system. The equivalence used to estimate K_{eff} and ξ_{eff} assumes that the displacement of the original structure is the same as that of the substitute structures. For inelastic systems, the effective stiffness may be assumed as the secant stiffness at some given displacement, while effective damping, which is utilized to qualify the energy dissipation, is assumed as the equivalent viscous damping. Substitute models are inadequate to assess local response of structures, although they are effective for global analyses.

13.5.2 STICK MODELS

These consist of multi-degree of freedom (MDOF) systems in which each element idealizes a number of members to the prototype structure. In multi-storey building frames, each storey is modeled by a single line of finite element representing the deformational characteristics of all columns and their interaction with beams. For 3D models, the stick elements relate the shear forces along two horizontal orthogonal directions and the storey torque to the corresponding inter-storey translation and rotation. The lateral stiffness of each equivalent stick element is the stiffness of the frame comprising columns and connected beams. For dynamic analysis, the mass of each floor is concentrated at the nodes representing the centroid of the slab. Lumping both mass and stiffness at a limited number of nodes and pairs of nodes leads to a significant reduction in the size of the problem to be solved. Distributed masses are seldom employed for stick models. They are used, for example, to simulate the response of structural walls. Shear beam elements are also utilized as stick elements for multi-storey frames employing members where shear deformation cannot be ignored. Stick models are suitable for sensitivity analyses to assess the effect of various parameters, such as beam-to-column strength ratio and the degree of irregularity along the height.

13.5.3 DETAILED MODELS

These include general FE idealization in which structures are discretized into a large number of elements with section analysis or spatial 2D or 3D. Such a modeling approach allows representation of details of the geometry of the members, and enables the description of the history of stresses and strains at fibers along the length or across the section dimensions. Provided that the problem size remains manageable, detailed models also provide global response quantities and the relationships between local and global response. In the detailed modeling approach, beams and columns of frames are represented by flexural elements, braces by truss elements and shear and core walls by 2D elements, such as plates and shells. For accurate evaluation of deformation and member forces, 3D modeling may be required. Its use is essential to study stress concentrations, local damage pattern or interface behavior between different materials. However, spatial FE models are often cumbersome for large structures, especially when inelastic dynamic analysis with large displacement is required.

Generic characteristics of the three levels of structural modelling mentioned above are summarized in **Table 13.5.** Their comparison is useful for the selection of an appropriate method of discretization while considering the objectives of the analysis, the accuracy desired and the computational resources available.

Table 13.5 Comparisons of Structural Models

Model Type	Type	3D Effects	Structure Prototype	Analysis Target	Complexity/ Accuracy	Computational Demand
Substitute	SDOF	-	Primarily regular structures	Global response	Low	Low
Stick	MDOF	√	All types of structure	Global response	Medium	Medium
Detailed	MDOF	√	All types of structure	Local and global response	High	High

Substitute and detailed models used to discretize structural system may be described as macro-and-micro models. Stick models constitute an intermediate group and employ member-level representation. Hybrid models, e.g. combining detailed and stick elements, can also be used especially for the seismic analysis of large structures. For example, the upper deck of multi-span bridges, which is expected to remain elastic, is often discretized using beam elements, while fine FE meshes are utilized for piers, where inelasticity is expected. For buildings, detailed models are often used to idealize the frame of the superstructure, while stick models are used for foundations.

13.6 PARAMETRIC STUDY

In "Handbook on Composite Construction of Multi-Storey Building" by Institute for Steel Development and Growth, design of a G+3 storied residential building **(Figs. 13.5 and 13.6)** using composite construction is given.

The same structure is analyzed here by using STAAD.Pro v8i software considering worst combination of probable loads, both normal dead and live loads and occasional loads like earthquake. Earthquake loads compatible to those of Kolkata (zone III) are considered in the analysis. For earthquake load calculations, IS:1893 -2002 is followed. As per the stipulations of National Building Code, maximum wind and maximum earthquake loads are not considered to act simultaneously. So, here only earthquake load is considered.

The reinforcement steel used conforms to Fe 415 as per IS: 1786 [115] while structural steel conforms to $f_y = 250$ N/mm^2 as per IS: 2062, Grade B (weldable). Concrete used conforms to Grade M20 as per IS 456: - 2000 satisfying durability requirements in Kolkata.

It has been observed that if the building is regular in plan and vertical bracings are provided along the cross direction, then the building becomes more cost effective. It is because of the enhanced structural stiffness and distribution of reactive forces mostly as axial tension or compression, which always has a advantage over frames without bracings.

13.6.1 Design Considerations

Place of construction	: Kolkata
Built-up area	: 80.9 m^2 each flat (2 flats on each floor)
Storey height	: 3.0 m
Safe S.B.C	: 7.5 t/m^2
Grade of concrete	: M 25
Dead load	: Partition walls and other external wall, floor finish as per IS: 875- 1987 [116]
Imposed load	: On floors of bedroom, kitchen, dining etc. 2 kN/m^2
	: On corridor, balcony and staircase 3 kN/m^2
	: Roof 1.5 kN/m^2
Density of Concrete (Dry)	: 2400 kg/m^3
Density of Concrete (Wet)	: 2500 kg/m^3
Concrete design	: IS: 456-2000
Steel design	: IS: 800-2007
Composite design	: IS: 11384-1985
Seismic Analysis	: IS: 1893-2002
Thickness of Deck	: 1.0 mm
Trough Spacing	: 306 mm
Deck slab definition	: One way floor load
	: Rib width: 0.18 m and rib height: 0.05 m

13.6.2 General Arrangement and Considerations of Building

- Floor shall be made of reinforced cement concrete with steel deck acting as form work and bottom reinforcement, with topping for floor finish.
- All beams, columns and bracings shall be made of steel.
- Beams are connected with columns considering simple connections. The stability of structural frame has been ensured by providing longitudinal and transverse bracings acting in unison with composite flooring forming a horizontal diaphragm.

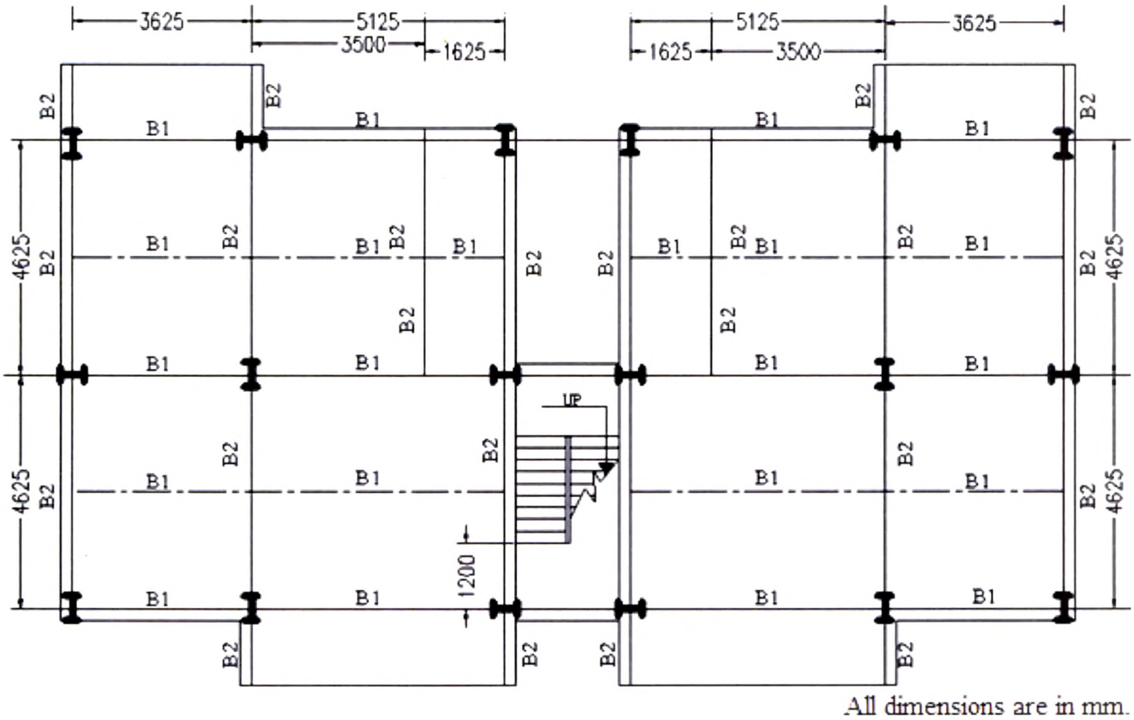


Fig. 13.5 Typical Floor Plan of a Structure [117]

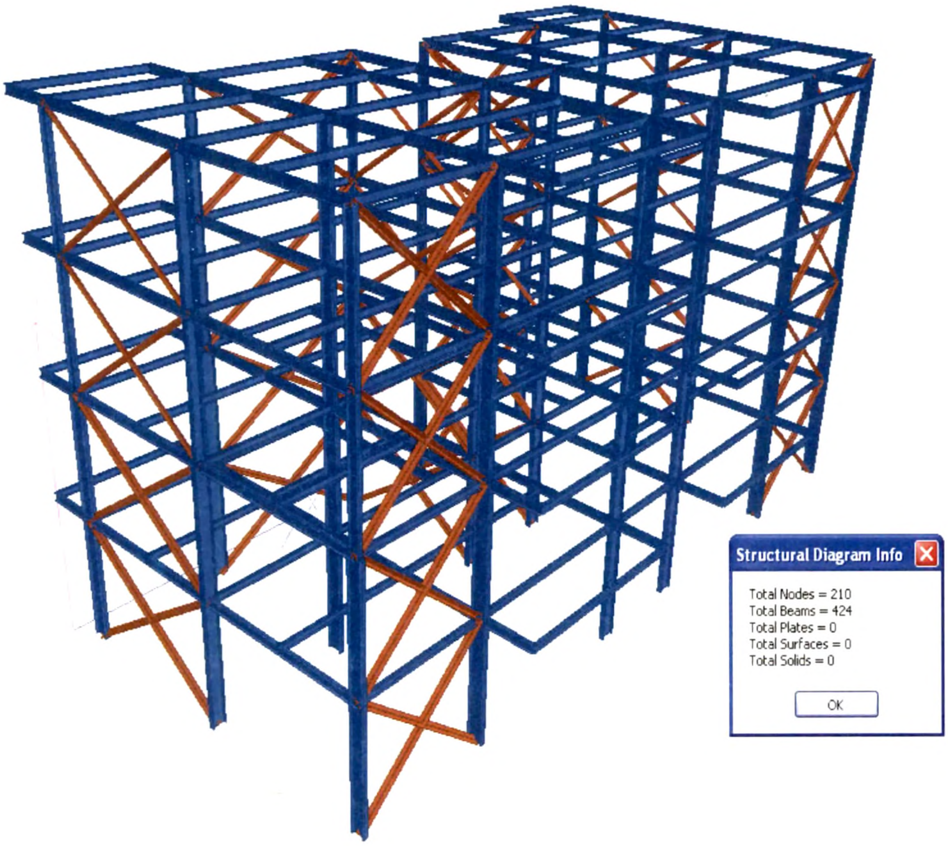


Fig. 13.6 A 3-D View of a Building with Structural Diagram Information

13.6.3 Analysis and Design of a Structure with STAAD.Pro V8i

There are four steps to reach the goal

i. Prepare input file
ii. Send the input file to the analysis/design engine.
iii. Read the results and verify them.
iv. Send the analysis result to either the concrete or steel design engines for designing. Then read and verify the design results.

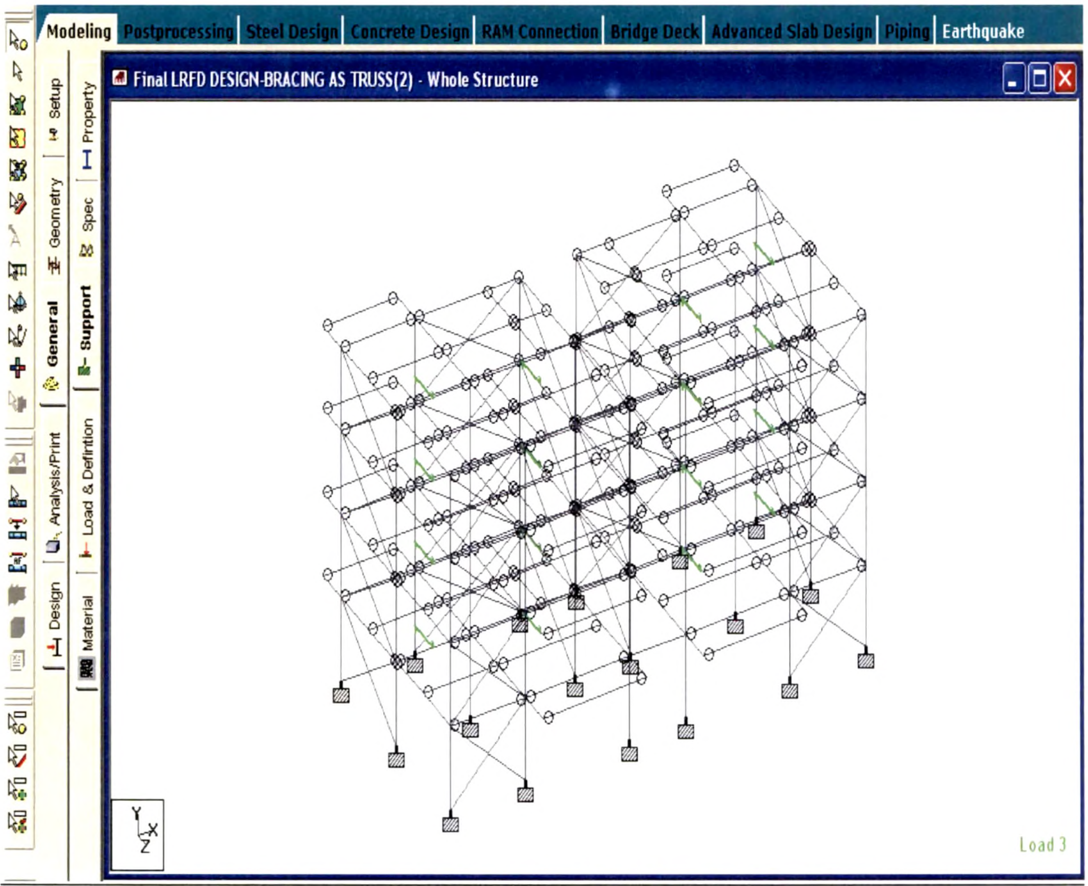


Fig. 13.7 Modelling Mode of STAAD.Pro V8i

Creating input file takes place in the modelling mode. This includes the geometry, the cross-sections, the material and geometric constants, support conditions, and finally the loading. To create the above model, steps are as follows:

Step 1: To create the geometry of a structure, the pre-processor of STAAD.Pro which comes with intelligent, accurate, speedy, error-free, and graphical methods is used. The pre-processor not only generates the geometry as shown in **Fig. 13.7** but also displays beam numbers as shown in **Fig. 13.8** and node numbers as shown in **Fig.13.9**.

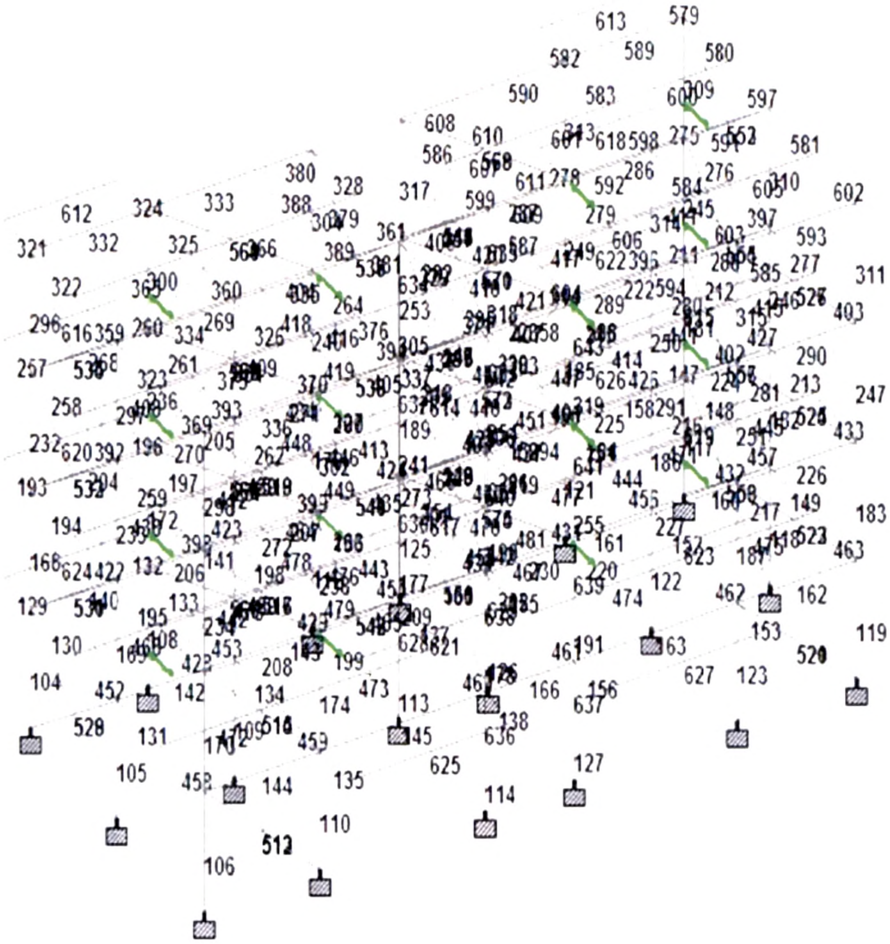


Fig. 13.8 Beam Number Display by GUI in Preprocessor

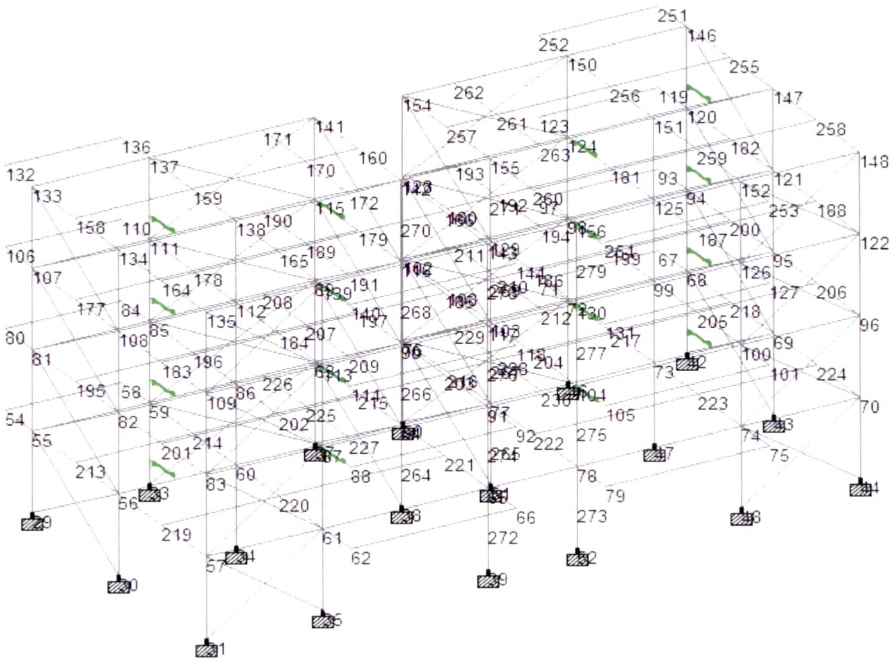


Fig 13.9 Node Number Display by GUI in Preprocessor

Step 2: From Generate toolbar, select Translation Repeat, or from menus select Geometry/Translation Repeat **(Fig. 13.10)**. The following dialog box will appear.

- ➢ Specify the Global Direction: select Y
- ➢ Specify the number of Steps.
- ➢ Specify Link Steps.

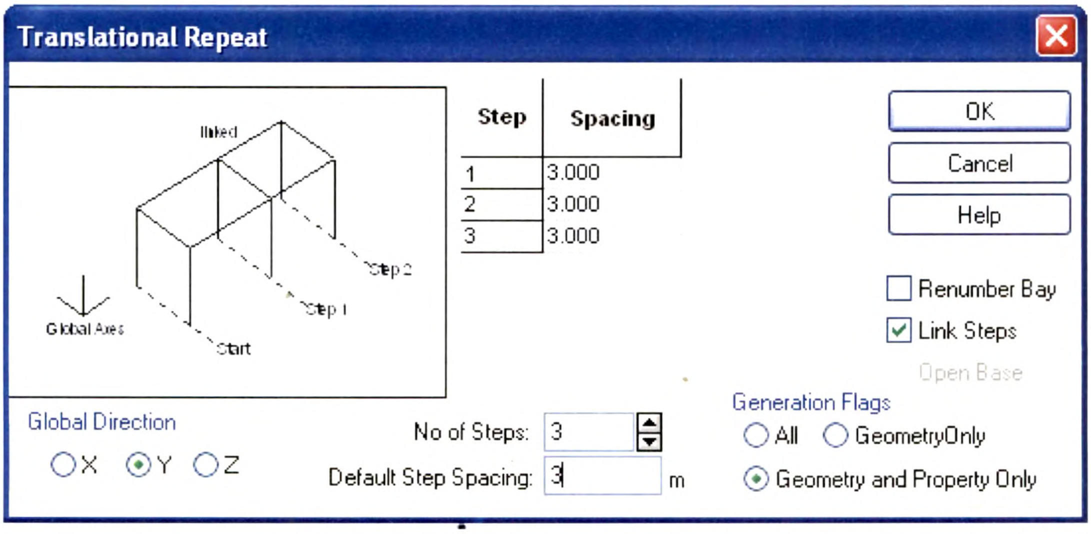


Fig. 13.10 Translational Repeat Screen

Step 3: Define Deck using the menu depicted in **Fig.13.11**

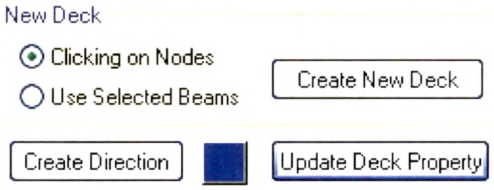
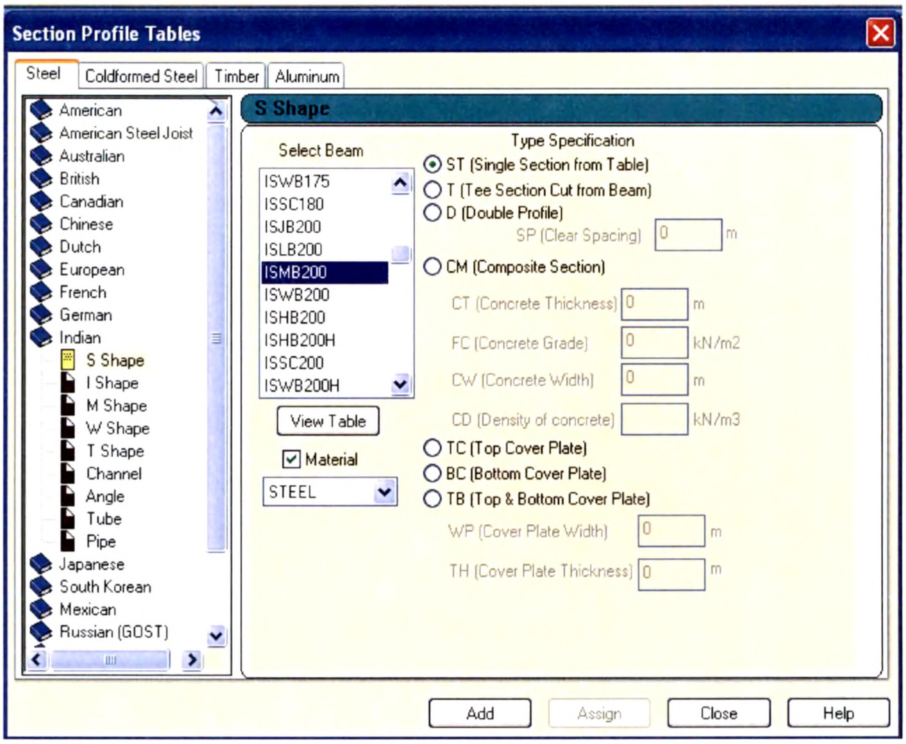


Fig 13.11 Creation of a New Deck

Step 4: After the creation of geometry, the next step is to Assign Properties to each beam and plate. For this, to use the Built in steel Table,

- Go to General page control
- From the property subpage, select the desired Steel Beam**(Fig.13.12)**.
- From Data Area at right of the screen, click Section Database.
- Select the type, size and specification.



. **Fig 13.12 Section Profile Table Dialog Box**

Step 5: Provide beam member specification as required using the menu depicted in **Fig. 13.13**.

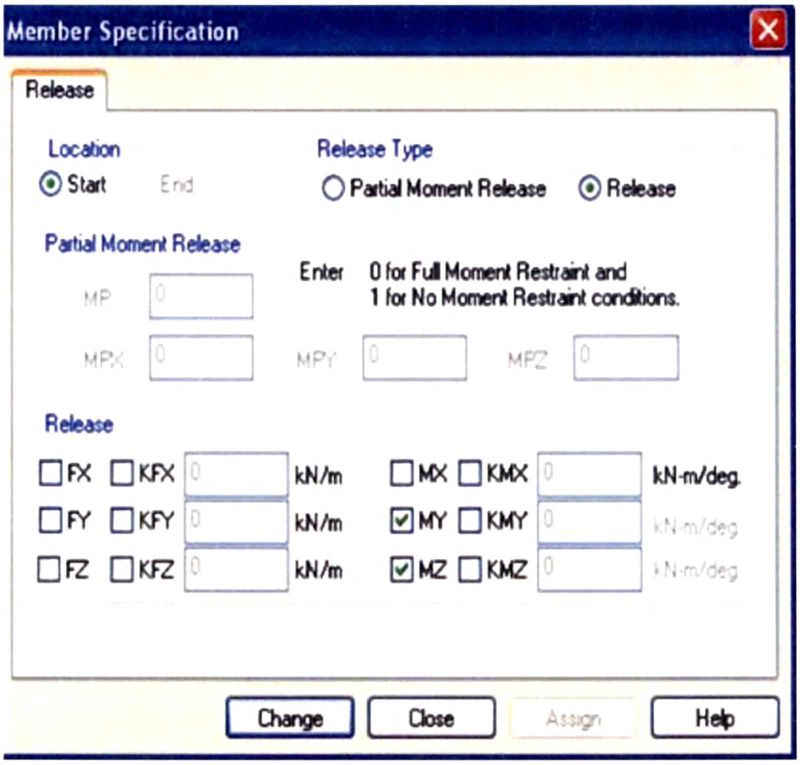


Fig. 13.13 Beam Member Specifications

Step 6: Provide support from the number of support condition options available.

Step 7: Loading is considered to be the last step in creating the input file before the Analysis command. A number of options are availablefor defining the load as shown in **Fig.13.14**.

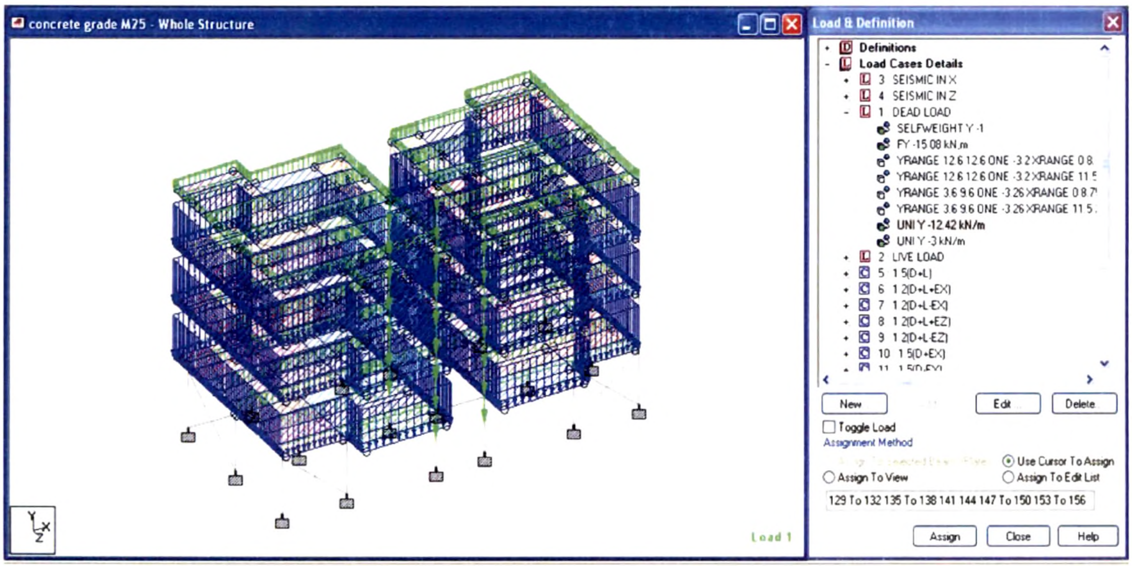


Fig. 13.14 Load and Definition Details of a Model

Step 8: Send the input file to Analysis/Design as shown in the screen shot of **Fig. 13.15**.

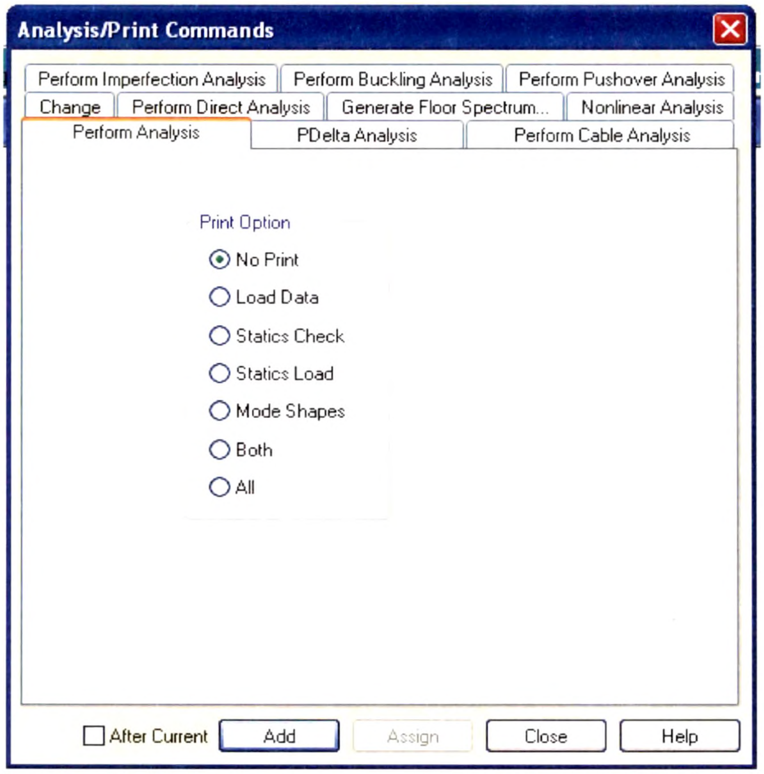
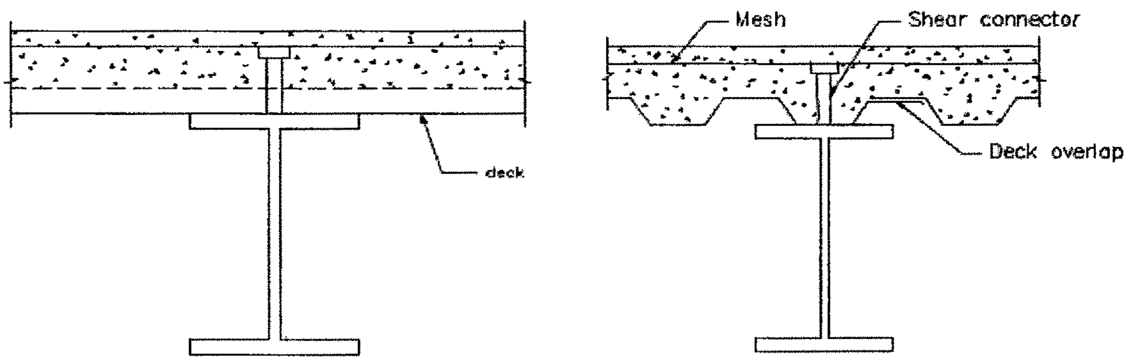


Fig. 13.15 Perform Analysis Screen

13.6.4 ORIENTATION OF DECK RIB

Following are the categories of composite beams in the AISC specifications, each with a differing effective concrete area.

Deck Perpendicular to Beam **(Fig. 13.16 (a))**



(a) Deck Ribs Perpendicular to Beam (b) Deck Ribs Parallel to Beam

Fig. 13.16 Orientation of Deck Rib with Respect to Beam

i. As illustrated in **Fig. 13.16 (a)**, concrete below the top of steel decking shall be neglected in computations of section properties and in calculating the number of shear studs, but the concrete below the top flange of deck may be included for calculating the effective width.

ii. The maximum spacing of shear connectors shall not exceed 32 in. (813 mm) along the beam length.

iii. The steel deck shall be anchored to the beam either by welding or by other means at a spacing not exceeding 16 in. (406 mm).

iv. A reduction factor as given by AISC formula

$$\left(\frac{0.85}{\sqrt{N_r}}\right)\left(\frac{0.85}{h_r}\right)\left(\frac{H_s}{h_r}-1.0\right) \leq 1.0 \qquad \text{...(13.4)}$$

should be used for reducing the allowable horizontal shear capacity of stud connectors. In the above formula h_r is the nominal rib height in inches; H_s is length of stud connector after welding in inches. An upper limit of (h_r + 3) is placed on the length of shear connectors used in computations even when longer studs are installed in metal decks. N_r is the number of studs in one rib; a maximum value of 3 can be used in computations although more than three studs may be installed and w_r is average width of rib.

<u>Deck Ribs Parallel to Beam</u> **Fig. 13.16 (b)**

1. The major difference between perpendicular and parallel orientation of deck ribs is that when the deck is parallel to beam, the concrete below the top of the decking can be included when calculating the number of shear studs, as shown in **Fig. 13.16 (b).**
2. If steel deck ribs occur on supporting beam flanges, it is permissible to cut high-hat to form a concrete haunch.
3. When the nominal rib height is 38.1 mm or greater, the minimum average width of deck flute should not be less than 51 mm for the first stud in the transverse row plus four stud diameters for each additional stud. This gives maximum average width of 51 mm for one stud, 51 mm plus 4d for two studs, 51 mm plus 8d for three studs etc., where d is the diameter of stud. Note that if a metal deck cannot accommodate this width requirement, the deck can be split over the girder from a haunch.
4. A reduction factor given by AISC formula

$$0.6\left(\frac{w_r}{h_r}\right)\left(\frac{H_s}{H_r} - 1.0\right) \leq 1.0 \qquad \dots(13.5)$$

shall be used for reducing the allowable horizontal shear capacity of stud connectors.

13.6.5 EFFECT OF CHANGE OF PROFILE SHEETS

Profiled steel sheets as sacrificial shuttering is a concept widely accepted in recent times for fast track construction. Its advantage is not only techno-economical but also highly practical and utilitarian. **Table 13.6** shows analysis results of variation in rib properties of metal deck.

Table 13.6 Analysis Results of Variation in Rib Properties of Metal Deck

Node Displacements									
Max/Min	USING INSDAG DATA			VULCRAFT_1.5VL			VERCO_N		
	Horz.	Vert.	Horz.	Horz.	Vert.	Horz.	Horz.	Vert.	Horz.
	X mm	Y mm	Z mm	X mm	Y mm	Z mm	X mm	Y mm	Z mm
Max X	5.36	-5.00	0.26	5.37	-5.08	0.26	6.16	-5.16	0.33
Min X	-5.21	-3.31	0.07	-5.22	-3.36	0.07	-5.99	-3.44	0.10
Max Y	0.02	0.54	1.77	0.02	0.55	1.78	4.01	0.61	0.13
Min Y	0.03	-16.85	0.43	0.03	-17.92	0.44	0.05	-17.87	0.50
Max Z	-0.05	-2.63	4.04	-0.05	-2.66	4.07	-0.10	-2.59	4.63
Min Z	-0.09	-2.55	-3.36	-0.10	-2.55	-3.39	-0.08	-2.72	-3.80

Support Reactions									
Max/Min	USING INSDAG DATA			VULCRAFT_1.5VL			VERCO_N		
	Horz	Vert.	Horz.	Horz.	Vert.	Horz.	Horz.	Vert.	Horz.
F	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	130.07	962.26	-14.33	129.92	962.65	-14.57	151.03	1076.67	-16.67
Min Fx	-129.09	965.38	-14.41	-128.95	965.83	-14.66	-150.03	1080.19	-16.80
Max Fy	0.30	1295.84	0.55	0.29	1296.09	0.59	0.44	1476.47	-0.05
Min Fy	-30.86	-165.94	-0.25	-30.85	-166.37	-0.24	-36.98	-192.21	-0.32
Max Fz	-62.34	953.15	123.33	-62.19	953.84	123.59	-71.18	1057.91	143.05
Min Fz	-0.11	998.06	-107.64	-0.11	999.64	-107.92	-0.15	1121.84	-122.67

Support Moments									
Max/Min. M	USING INSDAG DATA			VULCRAFT_1.5VL			VERCO_N		
	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	27.88	0.00	-0.03	28.04	0.00	-0.03	33.85	0.00	-0.02
Min Mx	-27.20	0.00	0.03	-27.38	0.00	0.03	-32.27	0.00	0.03
Max My	0.40	0.00	-24.34	0.43	0.00	-24.34	-0.12	0.00	-29.03
Min My	0.40	0.00	24.65	0.43	0.00	24.65	-0.12	0.00	29.36
Max Mz	0.40	0.00	24.65	0.43	0.00	24.65	-0.12	0.00	29.36
Min Mz	0.40	0.00	-24.34	0.43	0.00	-24.34	-0.12	0.00	-29.03

Beam End Forces									
Max/Min F	USING INSDAG DATA			VULCRAFT_1.5VL			VERCO_N		
	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	1295.84	-0.30	0.55	1296.09	-0.29	0.59	1476.47	-0.44	-0.05
Min Fx	-140.55	-0.25	-0.13	-140.98	-0.24	-0.13	-161.78	-0.32	-0.16
Max Fy	0.00	126.06	0.00	0.00	126.32	0.00	0.00	127.20	0.00
Min Fy	0.00	-143.35	0.00	0.00	-143.45	0.00	0.00	-144.25	0.00
Max Fz	839.12	0.21	2.11	839.52	0.22	2.18	890.22	0.21	1.28
Min Fz	214.09	0.01	-1.80	214.13	0.00	-1.84	213.91	0.08	-1.90

Beam End Moments									
Max/Min M	USING INSDAG DATA			VULCRAFT_1.5VL			VERCO_N		
	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	39.07	0.00	0.00	38.83	0.00	0.00	36.61	0.00	0.00
Min Mx	-39.17	0.00	0.00	-38.91	0.00	0.00	-36.54	0.00	0.00
Max My	0.00	2.90	-0.39	0.00	2.99	-0.39	0.00	1.86	0.13
Min My	0.00	-3.01	0.25	0.00	-3.10	0.27	0.00	-1.76	0.20
Max Mz	0.00	0.00	149.15	-0.01	0.00	149.44	0.07	0.00	152.88
Min Mz	-15.20	0.00	-143.20	-15.24	0.00	-143.26	-14.04	0.00	-143.56

Here, the only change between above models is the definition of rib properties. In the software one can only provide Rib Height and Rib Width. The thickness of metal sheet cannot be defined which is one of the drawbacks of this software. And, this is the one drawback of this software. It is clear from the **Table 13.6** that there is not much variation in values of Nodal Displacements, Support Reactions, Support Moments, Beam End forces and Beam End moments.

13.6.6 DESIGN PARAMETERS OF DIFFERENT COUNTRY CODES

13.6.6.1 Design Parameters of Indian Code (IS: 800-2007 LSD)

Table 13.7 Design Parameters According to IS: 800-2007 LSD

Parameter Name	Default Value	Description
FYLD	250 MPa	Yield strength of steel in current units.
FU	420 MPa	Ultimate tensile strength of steel in current units.
KY	1.0	K value in local Y-axis. Usually, the Minor Axis.
KZ	1.0	K value in local Z-axis. Usually, the Major Axis.
LY	Member Length	Length to calculate Slenderness Ratio for buckling about local Y axis.
LZ	Member Length	Same as above except in Z-axis (Major).
MAIN	180	Allowable Slenderness Limit for compression member (as per Section 3.8)
TMAIN	400	Allowable Slenderness Limit for tension member (as per Section 3.8)
NSF	1.0	Net Section Factor for tension member. A factor, based on the end-connection type, controlling the rupture strength of the net section
ALFA	0.8	0.6 = For one or two bolts 0.7 = For three bolts 0.8 = For four or more bolts (as per Section 6.3.3)
DBS	0.0	Check for design against block shear – 0.0 = Design against block shear will not be performed. 1.0 = Design against block shear will be performed. If DBS = 1.0, Non-Zero positive values of AVG, AVN,

		ATG and ATN must be supplied to calculate block shear strength Tdb.
AVG	None (Mandatory for block shear check)	Minimum gross area in shear along bolt line parallel to external force. This parameter is applicable only when DBS = 1.0 (as per Section 6.4.1).
AVN	None (Mandatory for Block Shear check)	Minimum Net Area in shear along bolt line parallel to external force. This parameter is applicable only when DBS = 1.0 (as per Section 6.4.1).
ATG	None (Mandatory for block shear check)	Minimum gross area in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of the force. This parameter is applicable only when DBS = 1.0 (as per Section 6.4.1).
ATN	None (Mandatory for Block Shear check)	Minimum net area in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of the force. This parameter is applicable only when DBS = 1.0 (as per Section 6.4.1).
KX	1.0	Effective length factor for lateral torsional buckling (as per Table-15, Section 8.3.1)
LX	Member Length	Effective length for lateral torsional buckling (as per Table-15, Section 8.3.1)
CAN	1.0	Beam Type, (as per section 8.2.1.2) 0.0 = Cantilever beam 1.0 = Non-Cantilever beam
PSI	1.0	Ratio of the moments at the ends of the laterally unsupported length of the beam = 0.8 : where factored applied moment and tension can vary independently = 1.0 : For any other case. (as per Section 9.3.2.1)
CMY CMZ CMX	0.9	CM value in local Y and Z axes (as per Section 9.3.2.2)
LAT	0.0	Equivalent uniform moment factor for lateral torsional buckling (as per Table 18, section 9.3.2.2) 0.0 = Beam is laterally unsupported

		1.0 = Beam is laterally supported (as per Section 8.2.1 and 8.2.2 respectively)
TRACK	0	Controls the levels of detail to which results are reported. 0 = Minimum detail. 1 = Intermediate detail level. 2 = Maximum detail.
RATIO	1.0	Permissible ratio of the actual to allowable stresses.
DFF	None (Mandatory for deflection check)	"Deflection Length" / Maximum allowable local deflection.
DJ1	Start joint of member	Joint No. denoting starting point for calculation of "Deflection Length".
DJ2	End joint of member	Joint No. denoting end point for calculation of "Deflection Length".

13.6.6.2 Design Parameters of American Code (AISC LRFD) [110]

The user can control the design through specification of proper parameters. The PROFILE parameter is available for only a limited number of codes like the AISC ASD and AISC LRFD. The user can specify up to three profiles (a1, a2 and a3). Profile is the first three letters of a section name from its steel table, like, W8X, W12, C10, L20 etc. The PROFILE parameter-name is used only for member selection where members are selected from each of those profile names. The PROFILE for a T-section is the corresponding W-shape. Also, the shape specified under PROFILE has to be the same as that specified initially under member properties.

The types of construction recognized by AISC specification have not changed, except that both "simple framing" (formerly Type 2) and "semi-rigid framing" (formerly Type 3) have been combined into the same category, Type PR (partially restrained). "Rigid Framing" (formerly Type 1) is now Type FR (fully restrained). Type FR construction is permitted unconditionally. Type PR construction may necessitate some inelastic, but self-limiting, deformation of a structural steel element. Thus, when specifying Type PR construction, the designer should take into consideration the effects of partial restraint on the stability of the structure, lateral deflections and second order bending moments. As stated in Section C1 of the LRFD specification, an analysis of second order effects is required. Thus, when using LRFD code for steel design, the user must use the P-Delta analysis feature of STAAD.

The interaction of flexure and axial forces in singly and doubly symmetric shapes is governed by interaction formulas which cover the general case of biaxial bending combined with axial force. They are also valid for uniaxial bending and axial force.

Shear strength as calculated in LRFD is governed by the following limit states: by yielding of the web; by inelastic buckling of the web and by elastic buckling of the web. Shear in wide flanges and channel sections are resisted by the area of the web, which is taken as the overall depth times the web thickness.

13.6.6.3 Design Parameters of Eurocode [7]

Table 13.8 Design Parameters According to EURO CODE

Parameter Name	Default Value	Description
KY	1.0	K factor in local y axis.
KZ	1.0	K factor in local z axis.
LY	Member Length	Compression length in local y axis, Slenderness ratio = (KY)*(LY)/(Ryy)
LZ	Member Length	Compression length in local z axis, Slenderness ratio = (KZ)*(LZ)/(Rzz)
UNL	Member Length	Unrestraint length of member used in calculating the lateral-torsional resistance moment of the member.
NSF	1.0	Net tension factor for tension capacity calculation.
SBLT	0.0	Indicates if the section is rolled or built-up. 0.0 = Rolled 1.0 = Built-up
CMM	1.0	Indicates type of loading on member. Can take a value from 1 to 8.
CMN	1.0	Indicates the level of End-Restraint. 1.0 = No fixity 0.5 = Full fixity 0.7 = One end free and other end fixed

DMAX	100.0 cm	Maximum allowable depth for the member.
DMIN	0	Minimum required depth for the member.
RATIO	1	Permissible ratio of loading to capacity.
CAN	0	Member will be considered as a cantilever type member for deflection checks. 0 - member will not be treated as a cantilever member 1 - the member will be treated as a cantilever member
GM0	1.1	Corresponds to the Γ_{m0} factor in EN 1993-1-1:2005
GM1	1.1	Corresponds to the Γ_{m1} factor in EN 1993-1-1:2005
GM2	1.1	Corresponds to the Γ_{m2} factor in EN 1993-1-1:2005
SGR	0	Steel grade as in table 3.1 of EN 1993-1-1:2005 0.0 - indicates S 235 grade steel 1.0 - indicates S 275 grade steel 2.0 - indicates S 255 grade steel 3.0 - indicates S 420 grade steel 4.0 - indicates S 460 grade steel
ZIV	0.8	Specifies a reduction factor for vectoral effects to be used in axial tension checks [Cl 5.5.3(2) of DD ENV 1993-1-1:1992]
C1	0	Corresponds to the C1 factor to be used to calculate Elastic critical moment Mcr as per Clause 6.3.2.2
C2	0	Corresponds to the C2 factor to be used to calculate Elastic critical moment Mcr as per Clause 6.3.2.2
C3	0	Corresponds to the C3 factor to be used to calculate Elastic critical moment Mcr as per Clause 6.3.2.2
MU	0	To be used with CMM values of 7 and 8.
ZG	+Section Depth/2	Distance of transverse load from shear Centre. Used to calculate Mcr.
KC	1.0	Corresponds to the correction factor as per Table 6.6 of EN 1993-1-1:2005. Program will calculate 'kc' automatically if this parameter is set to 0.

Load Combinations

Load Combinations for IS 800:2007

- 1.5(DL+L L)
- 1.2(DL+LL+EX)
- 1.2(DL+Ll-EX)
- 1.2(DL+LL+EZ)
- 1.2(DL+LL-EZ)
- 1.5(DL+EX)
- 1.5(DL-EX)
- 1.5(DL+EZ)
- 1.5(DL-EZ)
- 0.9DL+1.5EX
- 0.9DL-1.5EX
- 0.9DL+1.5EZ
- 0.9DL-1.5EZ

Load Combinations for Eurocode

- 1.35DL+ 1.5LL-0.7EX
- 1.35DL+ 1.5LL+0.7EX
- 1.35DL+ 1.5LL-0.7EZ
- 1.35DL+ 1.5LL+0.7EZ
- 0.925DL+1.5LL-0.7EX
- 0.925DL+ 1.5LL+0.7EX
- 0.925DL+ 1.5LL-0.7EZ
- 0.925DL+ 1.5LL+0.7EZ

Load Combinations for AISC LRFD

- 1.0DL
- 1.0DL+1.0LL
- 1.0DL+0.75LL
- 1.0DL+0.7EX
- 1.0DL-0.7EX
- 1.0DL+0.7EZ
- 1.0DL-0.7EZ
- 1.0DL+0.75LL+0.525EX
- 1.0DL+0.75LL+0.525EZ
- 1.0DL+0.75LL-0.525EX
- 1.0DL+0.75LL-0.525EZ
- 0.6DL
- 0.6DL+0.7EX
- 0.6DL+0.7EZ
- 0.6DL-0.7EX
- 0.6DL-0.7EZ
- 1.4DL
- 1.2DL+1.6LL+0.5EX
- 1.2DL+1.6LL+0.5EX

Table 13.9 Results obtained using Various Country Codes

Node Displacement									
Max/ Min	**IS 800 LSD**			**AISC LRFD**			**EUROCODE**		
	Horz.	Vert.	Horz.	Horz.	Vert.	Horz.	Horz.	Vert.	Horz.
D	X mm	Y mm	Z mm	X mm	Y mm	Z mm	X mm	Y mm	Z mm
Max X	5.36	-5.00	0.26	4.65	-0.59	-0.12	4.66	-0.67	-0.13
Min X	-5.21	-3.31	0.07	-3.24	-2.28	0.01	-3.31	-5.23	0.03
Max Y	0.02	0.54	1.77	4.47	0.65	0.32	0.15	0.72	3.18
Min Y	0.03	-16.85	0.43	1.43	-16.25	0.20	1.85	-18.97	-0.19
Max Z	-0.05	-2.63	4.04	0.06	0.40	2.98	0.13	0.50	3.18
Min Z	-0.09	-2.55	-3.36	-0.02	-1.77	-1.99	-0.05	-3.87	-2.14

Support Reactions									
Max/ Min	**IS 800 LSD**			**AISC LRFD**			**EUROCODE**		
F	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	130.07	962.26	-14.33	78.56	649.01	-9.53	99.07	709.76	-1.28
Min Fx	-129.09	965.38	-14.41	-77.65	651.82	-9.58	-98.06	712.96	-1.29
Max Fy	0.30	1295.84	0.55	1.86	1072.12	0.25	0.03	1177.00	-2.28
Min Fy	-30.86	-165.94	-0.25	-31.84	-163.83	-0.34	-34.05	-161.85	-0.04
Max Fz	-62.34	953.15	123.33	-33.63	589.45	64.75	0.07	771.08	77.70
Min Fz	-0.11	998.06	-107.64	0.05	866.19	-59.18	0.00	707.92	-83.52
Support Moments									
Max/ Min	**IS 800 LSD**			**AISC LRFD**			**EUROCODE**		
M	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	27.88	0.00	-0.03	15.23	0.00	-0.01	11.17	0.00	0.04
Min Mx	-27.20	0.00	0.03	-12.65	0.00	0.00	-13.87	0.00	0.00
Max My	0.40	0.00	-24.34	-0.30	0.00	-0.20	-1.68	0.00	-0.40
Min My	0.40	0.00	24.65	0.43	0.00	0.28	-1.77	0.00	0.41
Max Mz	0.40	0.00	24.65	-0.06	0.00	10.77	-0.04	0.00	9.87
Min Mz	0.40	0.00	-24.34	0.19	0.00	-7.62	0.09	0.00	-7.02
Beam End Forces									
Max/ Min	**IS 800 LSD**			**AISC LRFD**			**EUROCODE**		
F	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	1295.84	-0.30	0.55	1072.12	-1.86	0.25	1177.00	-2.28	0.03
Min Fx	-140.55	-0.25	-0.13	-137.66	-0.34	-0.16	-134.95	1.50	-0.04
Max Fy	0.00	126.06	0.00	0.00	105.74	0.00	0.00	110.60	0.00
Min Fy	0.00	-143.35	0.00	0.00	-120.84	0.00	0.00	-129.52	0.00
Max Fz	839.12	0.21	2.11	554.10	0.19	0.66	556.81	0.01	4.61
Min Fz	214.09	0.01	-1.80	0.68	0.04	-0.72	420.17	0.07	-2.93
Beam End Moments									
Max/ Min	**IS 800 LSD**			**AISC LRFD**			**EUROCODE**		
M	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	39.07	0.00	0.00	36.17	0.00	0.00	40.09	0.00	0.00
Min Mx	-39.17	0.00	0.00	-36.25	0.00	0.00	-40.00	0.00	0.00
Max My	0.00	2.90	-0.39	0.00	0.98	-0.30	0.00	6.92	-0.07
Min My	0.00	-3.01	0.25	0.00	-1.07	0.28	0.00	-6.91	-0.03
Max Mz	0.00	0.00	149.15	0.00	0.00	126.66	0.00	0.00	135.78
Min Mz	-15.20	0.00	-143.20	-7.05	0.00	-116.40	-0.93	0.00	-132.86

In the above comparison the design criteria for composite beam remains same as in the software we have the facility to design composite beam with AISC is given. While, other criteria like load combination, design for column, deflection check etc. can be considered according to the respective code. One has a facility for primary and secondary analysis in the software, but, to maintain the harmony, 'perform analysis' command is used in all the above cases.

The comparison of results obtained for nodal displacement, support reactions, support moments, beam end forces and beam end moments, different codes, is given in **Table 13.9.**

13.6.7 LSD Versus ASD Method

A comparison of results obtained using with new IS:800-2007 code (Limit State Design) with the old code (Allowable Stress Design) having gravity and seismic loading is given here in **Table 13.10**.

Table 13.10 Analysis Results using IS: 800 (LSD) and IS: 800 (ASD) with EQ Load

Node Displacements						
Max./Min.	IS 800 LSD			IS 800 ASD		
	Horizontal	Vertical	Horizontal	Horizontal	Vertical	Horizontal
D	X mm	Y mm	Z mm	X mm	Y mm	Z mm
Max X	5.36	-5.00	0.26	3.95	-4.04	-0.22
Min X	-5.21	-3.31	0.07	-3.84	-2.68	-0.21
Max Y	0.02	0.54	1.77	0.02	0.48	1.28
Min Y	0.03	-16.85	0.43	0.05	-15.67	0.06
Max Z	-0.05	-2.63	4.04	0.03	-3.08	2.79
Min Z	-0.09	-2.55	-3.36	-0.14	-3.09	-2.81
Support Reactions						
Max/Min F	IS 800 LSD			IS 800 ASD		
	Horizontal	Vertical	Horizontal	Horizontal	Vertical	Horizontal
	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	130.07	962.26	-14.33	143.37	1083.92	-18.39
Min Fx	-129.09	965.38	-14.41	-142.53	1088.13	-18.53
Max Fy	0.30	1295.84	0.55	-0.05	1482.02	0.01
Min Fy	-30.86	-165.94	-0.25	-35.19	-193.90	-0.25
Max Fz	-62.34	953.15	123.33	-66.37	1092.12	144.87
Min Fz	-0.11	998.06	-107.64	-0.17	1121.68	-116.61

Support Moments						
Max/Min M	IS 800 LSD			IS 800 ASD		
	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	27.88	0.00	-0.03	43.44	0.00	-0.10
Min Mx	-27.20	0.00	0.03	-39.34	0.00	-0.04
Max My	0.40	0.00	-24.34	2.41	0.00	-0.90
Min My	0.40	0.00	24.65	2.53	0.00	0.90
Max Mz	0.40	0.00	24.65	0.00	0.00	32.73
Min Mz	0.40	0.00	-24.34	0.01	0.00	-32.33
Beam End Forces						
Max/Min F	IS 800 LSD			IS 800 ASD		
	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	1295.84	-0.30	0.55	1482.02	0.05	0.01
Min Fx	-140.55	-0.25	-0.13	-165.04	-0.25	-0.28
Max Fy	0.00	126.06	0.00	0.00	129.43	0.00
Min Fy	0.00	-143.35	0.00	0.00	-144.17	0.00
Max Fz	839.12	0.21	2.11	896.61	-0.74	1.41
Min Fz	214.09	0.01	-1.80	213.98	0.04	-1.67
Beam End Moments						
Max/Min M	IS 800 LSD			IS 800 ASD		
	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	39.07	0.00	0.00	35.42	0.00	0.00
Min Mx	-39.17	0.00	0.00	-35.52	0.00	0.00
Max My	0.00	2.90	-0.39	0.00	2.55	-0.19
Min My	0.00	-3.01	0.25	0.00	-2.38	1.01
Max Mz	0.00	0.00	149.15	-0.20	0.00	149.34
Min Mz	-15.20	0.00	-143.20	-14.24	0.00	-144.26

13.6.8 Gravity Load (G.L.) Versus Gravity Load And Earthquake Load (E.L.)

Load combinations considered are such as to produce maximum forces and effects and consequently maximum stress and deformations. Also, from the codal provision, it is known that wind load and earthquake load shall not be assumed to act simultaneously. So, only earthquake load is considered while making comparison in **Tables 13.11** and **13.12**.

Table 13.11 Analysis Results of AISC LRFD with G.L. and G.L.+ E.L.

Node Displacements						
Max/Min D	**AISC LRFD ONLY GRAVITY LOAD**			**AISC LRFD (G.L.+E.L.)**		
	Horizontal	Vertical	Horizontal	Horizontal	Vertical	Horizontal
D	X mm	Y mm	Z mm	X mm	Y mm	Z mm
Max X	0.20	-3.53	0.76	4.65	-0.59	-0.12
Min X	-0.13	-3.62	0.69	-3.24	-2.28	0.01
Max Y	0.00	0.00	0.00	4.47	0.65	0.32
Min Y	0.10	-17.85	0.51	1.43	-16.25	0.20
Max Z	0.11	-5.52	1.05	0.06	0.40	2.98
Min Z	0.00	-0.93	-0.11	-0.02	-1.77	-1.99
Support Reactions						
Max/Min F	**AISC LRFD ONLY GRAVITY LOAD**			**AISC LRFD (G.L.+E.L.)**		
	Horizontal	Vertical	Horizontal	Horizontal	Vertical	Horizontal
	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	63.97	467.51	-6.66	78.56	649.01	-9.53
Min Fx	-62.88	469.69	-6.70	-77.65	651.82	-9.58
Max Fy	-0.05	854.51	0.19	1.86	1072.12	0.25
Min Fy	0.00	18.02	1.86	-31.84	-163.83	-0.34
Max Fz	-44.91	593.29	67.07	-33.63	589.45	64.75
Min Fz	-0.02	461.02	-53.51	0.05	866.19	-59.18
Support Moments						
Max/Min M	**AISC LRFD ONLY GRAVITY LOAD**			**AISC LRFD (G.L.+E.L.)**		
	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	8.66	0.00	-0.01	15.23	0.00	-0.01
Min Mx	-6.51	0.00	0.00	-12.65	0.00	0.00
Max My	-0.09	0.00	0.18	-0.30	0.00	-0.20
Min My	-0.09	0.00	-0.16	0.43	0.00	0.28
Max Mz	-0.09	0.00	0.18	-0.06	0.00	10.77
Min Mz	-0.09	0.00	-0.16	0.19	0.00	-7.62
Beam End Forces						
Max/Min F	**AISC LRFD ONLY GRAVITY LOAD**			**AISC LRFD (G.L.+E.L.)**		
	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	854.51	0.05	0.19	1072.12	-1.86	0.25
Min Fx	-14.46	-0.21	0.21	-137.66	-0.34	-0.16
Max Fy	0.00	82.41	0.00	0.00	105.74	0.00
Min Fy	0.00	-94.48	0.00	0.00	-120.84	0.00
Max Fz	554.53	0.18	0.41	554.10	0.19	0.66
Min Fz	164.94	-0.01	-0.30	0.68	0.04	-0.72

Beam End Moments						
Max/Min M	AISC LRFD ONLY GRAVITY LOAD			AISC LRFD (G.L.+E.L.)		
	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	29.90	0.00	0.00	36.17	0.00	0.00
Min Mx	-29.90	0.00	0.00	-36.25	0.00	0.00
Max My	0.00	0.62	-0.24	0.00	0.98	-0.30
Min My	0.00	-0.60	0.29	0.00	-1.07	0.28
Max Mz	0.00	0.00	98.75	0.00	0.00	126.66
Min Mz	-10.92	0.00	-96.19	-7.05	0.00	-116.40

Table 13.12 Analysis Results of IS: 800-2007 Code with G.L. and G.L. + E.L.

Node Displacements						
Max/Min	IS 800 LSD with G.L. + E.L.			IS 800 LSD with G.L.		
	Horizontal	Vertical	Horizontal	Horizontal	Vertical	Horizontal
D	X mm	Y mm	Z mm	X mm	Y mm	Z mm
Max X	5.36	-5.00	0.26	0.11	-4.20	0.43
Min X	-5.21	-3.31	0.07	-0.04	-3.65	0.70
Max Y	0.02	0.54	1.77	0.00	0.00	0.00
Min Y	0.03	-16.85	0.43	0.03	-16.85	0.43
Max Z	-0.05	-2.63	4.04	0.05	-4.61	1.01
Min Z	-0.09	-2.55	-3.36	0.01	-0.75	-0.10
Support Reactions						
Max/Min F	IS 800 LSD with G.L. + E.L.			IS 800 LSD with G.L.		
	Horizontal	Vertical	Horizontal	Horizontal	Vertical	Horizontal
	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	130.07	962.26	-14.33	89.89	763.72	-14.88
Min Fx	-129.09	965.38	-14.41	-88.31	766.43	-14.98
Max Fy	0.30	1295.84	0.55	0.30	1295.84	0.55
Min Fy	-30.86	-165.94	-0.25	0.00	22.17	2.09
Max Fz	-62.34	953.15	123.33	-65.41	860.04	83.95
Min Fz	-0.11	998.06	-107.64	0.13	952.66	-69.13
Support Moments						
Max/Min M	IS 800 LSD with G.L. + E.L.			IS 800 LSD with G.L.		
	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	27.88	0.00	-0.03	13.38	0.00	0.01
Min Mx	-27.20	0.00	0.03	-13.80	0.00	0.02
Max My	0.40	0.00	-24.34	0.54	0.00	-0.42
Min My	0.40	0.00	24.65	0.54	0.00	0.69
Max Mz	0.40	0.00	24.65	0.51	0.00	0.72
Min Mz	0.40	0.00	-24.34	0.51	0.00	-0.45

Beam End Forces						
Max/Min F	IS 800 LSD with G.L. + E.L.			IS 800 LSD with G.L.		
	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	1295.84	-0.30	0.55	1295.84	-0.30	0.55
Min Fx	-140.55	-0.25	-0.13	-31.19	-0.46	0.46
Max Fy	0.00	126.06	0.00	0.00	126.06	0.00
Min Fy	0.00	-143.35	0.00	0.00	-143.35	0.00
Max Fz	839.12	0.21	2.11	842.19	0.18	1.20
Min Fz	214.09	0.01	-1.80	251.20	-0.10	-0.87
Beam End Moments						
Max/Min M	IS 800 LSD with G.L. + E.L.			IS 800 LSD with G.L.		
	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	39.07	0.00	0.00	37.36	0.00	0.00
Min Mx	-39.17	0.00	0.00	-37.38	0.00	0.00
Max My	0.00	2.90	-0.39	0.00	1.84	-0.39
Min My	0.00	-3.01	0.25	0.00	-1.77	0.14
Max Mz	0.00	0.00	149.15	0.00	0.00	149.15
Min Mz	-15.20	0.00	-143.20	-15.20	0.00	-143.20

13.6.9 Effect Of Orientation of Column

The columns in a building usually carry axial compressive loads. Thus, compression members are subjected to loads that tend to decrease their lengths. Except in pin-jointed trusses, such members, under external loads experience bending moments and shear forces. If the net moments are zero, the compression member is required to resist the load acting concentric to the original longitudinal axis of the member and is termed as axially loaded column. If the net end moments are not zero, the members will be subjected to an axial load and bending moments along its length.

Buckling phenomenon is associated with the stiffness of member. A member with low stiffness will buckle early than one with high stiffness. Increasing member length will cause reduction in stiffness. The stiffness of member is strongly influenced by the amount and distribution of the material in the cross-section of column; the value of r reflects the way in which the material is distributed. Also, note that any member will tend to buckle about the weak axis. **Figure 13.17** shows the orientation of column parallel to x- and z- axes.

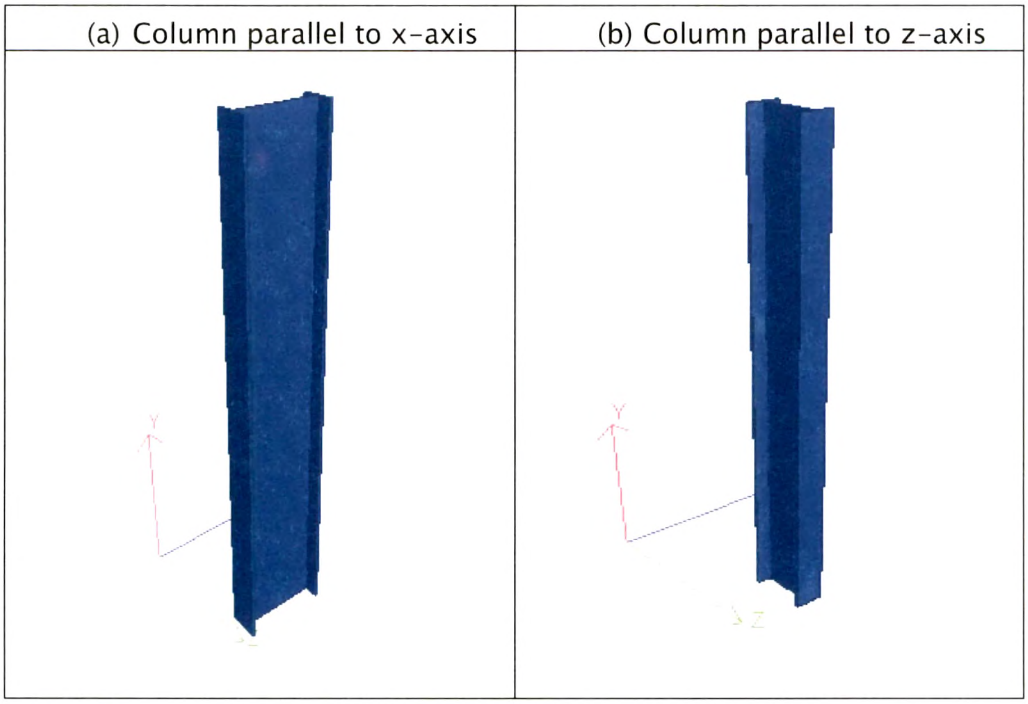


Fig. 13.17 Orientation of Column

Table 13.13 Comparison of Design of Beam No. 376

CODE FOR DESIGN	AISC LRFD	AISC LRFD
Loading criteria	Gravity + Seismic	Gravity + Seismic
Orientation of column	Parallel to X-Axis	Parallel to Z-Axis
Section	CM ISMB 150	CM ISMB 175
Stress ratio	0.586	0.431
Status	Pass	Pass
Section location	2.56 m	2.56 m
Load combination	1.0DL+1.0LL	1.0DL+1.0LL
Moment	- 60.096 kN-m	- 60.242 kN-m
Nominal moment capacity	102.47 kN-m	139.76 kN-m
Maximum shear force	46.950 kN	47.064 kN
No. of shear connector	14	14
Dia. of shear connector	19 mm	19 mm
Total steel weight of structure	36785.41 kg	31484.73 kg

I-sections are often used as columns in buildings. Though the r values of I-sections about the two axes are not same, they are better than those of channels. Since I-sections have thick flanges (which avoid the problem of local bucking) and are amenable for easy connections, they are often used as compression members in buildings.

Table 13.14 Analysis Results for Two Different Orientations of Column

Node Displacements						
Max/Min	COLUMN PARALLEL TO X-AXIS			COLUMN PARALLEL TO Z-AXIS		
	Horizontal	Vertical	Horizontal	Horizontal	Vertical	Horizontal
D	X mm	Y mm	Z mm	X mm	Y mm	Z mm
Max X	3.56	-0.21	-0.20	5.35	-0.55	-0.36
Min X	-2.47	-5.00	1.49	-3.71	-3.14	1.52
Max Y	0.05	0.70	2.59	-0.06	1.13	2.96
Min Y	0.00	-13.76	1.81	0.08	-14.88	1.51
Max Z	-0.16	-6.31	5.16	0.01	-2.93	4.91
Min Z	0.00	-0.33	-1.82	-0.13	-1.83	-1.61
Support Reactions						
Max/Min F	COLUMN PARALLEL TO X-AXIS			COLUMN PARALLEL TO Z-AXIS		
	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	46.66	448.71	3.45	76.41	530.02	2.09
Min Fx	-46.35	451.27	3.42	-75.76	531.95	2.06
Max Fy	-0.15	875.43	-50.28	0.01	806.27	-67.11
Min Fy	-28.05	-155.49	-0.02	-34.48	-171.68	-0.09
Max Fz	-21.40	501.66	40.56	-38.36	555.18	67.42
Min Fz	-0.18	822.02	-54.45	0.01	756.09	-69.58
Support Moments						
Max/Min M	COLUMN PARALLEL TO X-AXIS			COLUMN PARALLEL TO Z-AXIS		
	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	16.38	0.00	-0.33	17.88	0.00	0.01
Min Mx	-12.12	0.00	-1.51	-17.14	0.00	-0.01
Max My	14.45	0.00	-21.45	-12.65	0.00	-0.19
Min My	14.42	0.00	21.74	-0.21	0.00	0.27
Max Mz	-0.10	0.00	30.07	-0.57	0.00	0.47
Min Mz	15.34	0.00	-21.54	12.88	0.00	-0.35
Beam End Forces						
Max/Min F	COLUMN PARALLEL TO X-AXIS			COLUMN PARALLEL TO Z-AXIS		
	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	842.67	0.15	-0.18	786.20	14.99	0.01
Min Fx	-137.34	8.40	0.02	-144.15	0.04	-0.15
Max Fy	0.00	81.18	0.00	0.00	81.18	0.00
Min Fy	0.00	-77.73	0.00	0.00	-78.86	0.00
Max Fz	155.48	-0.33	30.97	129.42	27.96	0.64
Min Fz	90.89	0.03	-24.23	0.24	0.54	-0.88

Beam End Moments						
Max/Min M	COLUMN PARALLEL TO X-AXIS			COLUMN PARALLEL TO Z-AXIS		
	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	33.79	0.00	0.00	37.50	0.00	0.00
Min Mx	-33.99	0.00	0.00	-37.82	0.00	0.00
Max My	0.00	45.86	-0.88	0.00	0.85	0.75
Min My	0.00	-48.21	-0.89	0.00	-0.63	43.41
Max Mz	0.00	0.00	99.43	0.00	0.00	98.99
Min Mz	-0.12	0.00	-135.02	0.03	0.00	-130.19

Here, in the above comparison given in **Table 13.14**, instead of analysis and design criteria (like in the rest of comparison models), change is in the general criteria like specification. To change the orientation of column, 'beta angle' 90 degrees is used. To find out the difference in design of a particular beam, comparison of design for beam no. 376, which is critical, is given here in **Table 13.13**.

13.6.10 Effect of Change of Grade of Concrete

Different strength of concrete can be achieved by using different proportions of ingredients of concrete. One of the disadvantages of conventional concrete is the high self weight. Density of the normal concrete is of the order of 2200 kg/m^3 to 2600 kg/m^3. This heavy self weight will make it to some extent an uneconomical structural material. In light weight concrete, density varies from 300 kg/m^3 to 1850 kg/m^3. There are many advantages of having low density. It helps in reduction of dead load, increases the progress of building, and lowers haulage and handling cost. In framed structures, the beams and columns have to carry load of floors and walls. If floors and walls are made up of light weight concrete it will result in considerable economy. Another most important characteristic of light weight concrete is the relatively low thermal conductivity, a property which improves with low density. A concrete which is light in weight and sufficiently strong when used in conjunction with steel reinforcement will be a material which is more economical than the conventional concrete. Normal weight concrete results are compared with light weight concrete results in **Table 13.15.**

Table 13.15 Analysis Results of Normal Weight Concrete Versus Light Weight Concrete

Node Displacements						
Max/Min D	Normal Weight Concrete (25 kN/m^3)			Light Weight Concrete (15 kN/m^3)		
	Horizontal	Vertical	Horizontal	Horizontal	Vertical	Horizontal
	X mm	Y mm	Z mm	X mm	Y mm	Z mm
Max X	5.36	-5.00	0.26	4.66	-4.45	-0.10
Min X	-5.21	-3.31	0.07	-4.58	-1.64	0.28
Max Y	0.02	0.54	1.77	0.03	0.53	1.62
Min Y	0.03	-16.85	0.43	0.04	-17.94	0.30
Max Z	-0.05	-2.63	4.04	0.04	-3.56	3.56
Min Z	-0.09	-2.55	-3.36	-0.09	-2.37	-3.26
Support Reactions						
Max/Min F	Normal Weight Concrete (25 kN/m^3)			Light Weight Concrete (15 kN/m^3)		
	Horizontal	Vertical	Horizontal	Horizontal	Vertical	Horizontal
	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	130.07	962.26	-14.33	104.13	753.06	-11.58
Min Fx	-129.09	965.38	-14.41	-103.90	757.67	-11.64
Max Fy	0.30	1295.84	0.55	-0.25	1130.34	1.09
Min Fy	-30.86	-165.94	-0.25	-28.41	-143.36	-0.16
Max Fz	-62.34	953.15	123.33	-60.40	810.81	109.75
Support Moments						
Max/Min M	Normal Weight Concrete (25 kN/m^3)			Light Weight Concrete (15 kN/m^3)		
	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	27.88	0.00	-0.03	27.90	0.00	-0.01
Min Mx	-27.20	0.00	0.03	-19.29	0.00	0.01
Max My	0.40	0.00	-24.34	5.96	0.00	-0.74
Min My	0.40	0.00	24.65	6.17	0.00	0.75
Max Mz	0.40	0.00	24.65	1.20	0.00	20.80
Min Mz	0.40	0.00	-24.34	1.20	0.00	-20.69
Beam End Forces						
Max/Min F	Normal Weight Concrete (25 kN/m^3)			Light Weight Concrete (15 kN/m^3)		
	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	1295.84	-0.30	0.55	1130.34	0.25	1.09
Min Fx	-140.55	-0.25	-0.13	-119.95	-0.16	-0.08
Max Fy	0.00	126.06	0.00	0.00	110.93	0.00
Min Fy	0.00	-143.35	0.00	0.00	-122.83	0.00
Max Fz	839.12	0.21	2.11	717.40	-0.41	3.05
Min Fz	214.09	0.01	-1.80	172.79	0.04	-1.26

Beam End Moments						
Max/Min M	Normal Weight Concrete (25 kN/m^3)			Light Weight Concrete (15 kN/m^3)		
	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	39.07	0.00	0.00	44.04	0.00	0.00
Min Mx	-39.17	0.00	0.00	-44.03	0.00	0.00
Max My	0.00	2.90	-0.39	0.00	4.32	0.48
Min My	0.00	-3.01	0.25	0.00	-4.39	-0.74
Max Mz	0.00	0.00	149.15	0.00	0.00	137.89
Min Mz	-15.20	0.00	-143.20	-15.48	0.00	-121.10

13.6.11 RCC Structure versus Composite Structure

In many situations, lighter steel structures are invariably preferred to the heavier alternatives such as reinforced concrete or prestressed concrete. Steel also plays an important role in composite construction in conjunction with reinforced and prestressed concrete structures. Results obtained for a RCC structure are compared with those obtained using composite construction in **Table 13.16**.

Table 13.16 Comparison of Analysis Results - RCC versus Composite Structure

Node Displacements						
Max/Min	Composite Design With IS 800 LSD			Concrete Design with IS 13920		
	Horizontal	Vertical	Horizontal	Horizontal	Vertical	Horizontal
D	X mm	Y mm	Z mm	X mm	Y mm	Z mm
Max X	5.36	-5.00	0.26	13.82	-4.73	1.30
Min X	-5.21	-3.31	0.07	-13.73	-4.83	1.21
Max Y	0.02	0.54	1.77	0.04	1.24	4.83
Min Y	0.03	-16.85	0.43	0.03	-8.92	12.98
Max Z	-0.05	-2.63	4.04	0.22	-1.67	16.43
Min Z	-0.09	-2.55	-3.36	-0.06	-1.29	-13.70
Support Reactions						
Max/Min F	Composite Design With IS 800 LSD			Concrete Design with IS 13920		
	Horizontal	Vertical	Horizontal	Horizontal	Vertical	Horizontal
	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	130.07	962.26	-14.33	53.91	1453.91	-17.74
Min Fx	-129.09	965.38	-14.41	-57.07	1471.07	-17.73
Max Fy	0.30	1295.84	0.55	-4.35	1735.79	18.84
Min Fy	-30.86	-165.94	-0.25	-16.25	-65.35	1.55
Max Fz	-62.34	953.15	123.33	-2.69	1479.79	56.81
Min Fz	-0.11	998.06	-107.64	-13.41	1203.21	-57.51

Support Moments						
Max/Min M	Composite Design With IS 800 LSD			Concrete Design with IS 13920		
	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	27.88	0.00	-0.03	142.04	-0.02	-9.48
Min Mx	-27.20	0.00	0.03	-155.14	0.20	-14.87
Max My	0.40	0.00	-24.34	-7.70	0.33	63.32
Min My	0.40	0.00	24.65	-8.63	-0.35	-64.89
Max Mz	0.40	0.00	24.65	-23.60	0.31	106.10
Min Mz	0.40	0.00	-24.34	-23.69	-0.33	-102.82
Beam End Forces						
Max/Min F	Composite Design With IS 800 LSD			Concrete Design with IS 13920		
	Fx kN	Fy kN	Fz kN	Fx kN	Fy kN	Fz kN
Max Fx	1295.84	-0.30	0.55	1735.79	4.35	18.84
Min Fx	-140.55	-0.25	-0.13	-76.30	4.41	0.08
Max Fy	0.00	126.06	0.00	0.00	170.47	0.00
Min Fy	0.00	-143.35	0.00	0.00	-134.33	0.00
Max Fz	839.12	0.21	2.11	1081.11	-7.88	73.09
Min Fz	214.09	0.01	-1.80	1044.00	22.23	-68.96
Beam End Moments						
Max/Min M	Composite Design With IS 800 LSD			Concrete Design with IS 13920		
	Mx kNm	My kNm	Mz kNm	Mx kNm	My kNm	Mz kNm
Max Mx	39.07	0.00	0.00	21.52	0.00	52.49
Min Mx	-39.17	0.00	0.00	-22.01	0.00	-54.23
Max My	0.00	2.90	-0.39	0.20	155.14	-14.87
Min My	0.00	-3.01	0.25	-0.02	-142.04	-9.48
Max Mz	0.00	0.00	149.15	-1.90	0.00	212.91
Min Mz	-15.20	0.00	-143.20	-0.57	13.47	-107.48

A comparison of weight under different types of construction is reported here in **Table 13.17**.

Table 13.17 Total Weight of a Typical Structure

Type of Structure	Total Weight Kg.
Total weight of steel structure of normal weight concrete	37617
Total weight of steel structure of light weight concrete	32971
Total volume of concrete of reinforced concrete structure Bar Weight used in reinforced concrete structure	173880 5555
Total weight of composite structure designed by Eurocode	32770
Total weight of composite structure designed by AISC LRFD	33439
Total weight of composite structure designed by IS:800-2007	37617