STRUCTURAL ANALYSIS AND DESIGN OF AN INDUSTRIAL BUILDING
CHAPTER 1
EXECUTIVE SUMMARY
1. Executive summary

The aim of the project “ANALYSIS AND DESIGN OF INDUSTRIAL BUILDING” is to develop a Normal type of industrial building with steel roof structures on open frames considering the economy of the structure, as the part of project work under the curriculum of JNTU Hyderabad and Gokaraju Rangaraju Institute of Engineering and Technology, which fosters the creativity, designing skills, management skills and academic excellence.

The present scope of this project is to select a representative truss and analyse the truss for different load conditions possible at the project construction location.

This project design was assumed to be Industrial structure nearby Miyapur, Hyderabad.

The aim of this project is to design an industrial building economically using manual design techniques and computer aided design. The Project Summary Report emphasizes the structural analysis and design findings of industrial structure project.

The different phases of this design process include:

**PHASE 1:**
Phase one includes, drafting of representative truss in drafting software AutoCAD, developed by Autodesk.

**PHASE 2:**
Structural analysis of the steel truss, design of steel elements and reinforcement in concrete columns with the aid of STAAD Pro, developed by Bentley.

**PHASE 3:**
Design of RCC flat slabs, at plinth level, to carry live loads and dead loads.

**PHASE 4:**
- Design of footings including footings for columns from truss carrying roof loads, wind loads, earthquake loads and their combinations
- Footings for machines.

**PHASE 5:**
Detailing of various type of connections to be used within the truss.

**PHASE 6:**
The final stage in the project work is development of drawings using software.
CHAPTER 2
INTRODUCTION
2. Introduction:

2.1. Problem definition
Analyse and design an economical and stable roofed truss for the usage in industrial purposes like storage, workshops, warehouses, etc., using STAAD PRO and manual calculations.

2.2. Scope
The main scope of this project is to apply classroom knowledge in the real world by designing a roofed building. These buildings require large and clear areas unobstructed by the columns. The large floor area provides sufficient flexibility and facility for later change in the production layout without major building alterations. The industrial buildings are constructed with adequate headroom for the use of an overhead traveling crane.

2.3. General
Steel-framed buildings are commonly in use for industrial purposes. They are classified into three broad categories:

- Warehouse and factory buildings.
- Large span storage buildings.
- Heavy industrial process plant structures.

In the design of industrial buildings, load conditions and geometrical factors will dictate the degree of complication and hence the economy. The designer should possess good knowledge about the industrial process or purpose for which the building is intended. In this way, an optimum balance between safety, function, and economy can be achieved. The main dimensions of an industrial building are usually determined from a combination of functional and design considerations. Its width is derived first from an owner’s study of the space required to carry out the processing or storage operations. The designer then needs to consider whether this width can be provided economically by a single clear span, or whether multi-bay spans are feasible. Likewise, the overall length is usually readily determined by the owner, but the designer should give thought to the optimum bay length. Some of the factors affecting the choice are:

- Foundation conditions and their ability to accept the column loads.
- Crane runway girder considerations
- Purlin and girt capacities
- Masonry bond dimensions.
- Tilt-up concrete panel size and available carnage.

The building height is again a functional consideration, for buildings with overhead travelling Cranes the critical dimension is the clearance required under the hook. In Hyderabad, there is no snow and therefore fairly low roof pitches are practicable. The steeper the slope the better the structural action, but this benefit is usually outweighed by additional sheeting costs. In practice,
roof pitches between 5 and 10 are preferred. These pitches are suitable for any of the continuous length steel sheet roofing profiles, some of which are adequate for pitches down to 1.

2.4. Building configuration of a General Industrial structure

Typically the bays in industrial buildings have frames spanning the width direction. Several such frames are arranged at suitable spacing to get the required length. Depending upon the requirement, several bays may be constructed adjoining each other. The choice of structural configuration depends upon the span between the rows of columns, the head room or clearance required the nature of roofing material and type of lighting. If span is less, portal frames such as steel bents or gable frames can be used but if span is large then buildings with trusses are used.

Fig. 2.1. Typical structural layout of an industrial
The horizontal and vertical bracings, employed in single and multi-storey buildings, are also trusses used primarily to resist wind and other lateral loads. These bracings minimize the differential deflection between the different frames due to crane surge in industrial buildings. They also provide lateral support to columns in small and tall buildings, thus increasing the buckling strength.

Floors
Different types of floor are required in any factory from their use consideration such as production, workshop, stores, amenities, and administration. The service condition will vary widely in these areas, so different floors types are required. Industrial floors shall have sufficient resistance to abrasion, impact, acid action and temperatures depending on the type of activity carried out. High strength and high performance concretes can satisfy most of these requirements economically and is the most common material used. Foundation for vibrating machinery (such as reciprocating and high speed rotating machinery) should be placed upon rock or firm ground and it should be separated from adjacent floor to avoid vibrations.

Roof System
While planning a roof, designer should look for following quality lightness, strength, water proofness, insulation, fire resistance, cost, durability and low maintenance charges. Sheeting, purlin and supporting roof trusses supported on column provide common structural roof system for industrial buildings. The type of roof covering, its insulating value, acoustical properties, the appearance from inner side, the weight and the maintenance are the various factors, which are given consideration while designing the roof system. Brittle sheeting such as asbestos, corrugated and trafford cement sheets or ductile sheeting such as galvanized iron corrugated or profiled sheets are used as the roof covering material. The deflection limits for purlins and truss depend on the type of sheeting. For brittle sheeting small deflection values are prescribed in the code.

Lighting
Industrial operations can be carried on most efficiently when adequate illumination is provided. The requirements of good lighting are its intensity and uniformity. Since natural light is free, it is economical and wise to use daylight most satisfactorily for illumination in industrial plants whenever practicable. Side windows are of much value in lighting the interiors of small buildings but they are not much effective in case of large buildings. In case of large buildings monitors are useful (Fig. 2.2.).

Ventilation
Ventilation of industrial buildings is also important. Ventilation will be used for removal of heat, elimination of dust, used air and its replacement by clean fresh air. It can be done by means of natural forces such as aeration or by mechanical equipment such as fans. The large height of the roof may be used advantageously by providing low level inlets and high level outlets for air.
CHAPTER 3
DESCRIPTION OF ROOF TRUSS
3.1. Roof truss

Steel trusses are commonly used in commercial construction. They are pre-manufactured to order and are made in an open web design. They are essentially axially loaded members which are more efficient in resisting external loads since the cross section is nearly uniformly stressed. They are extensively used, especially to span large gaps. Trusses are used in roofs of single storey industrial buildings, long span floors and roofs of multistory buildings, to resist gravity loads.

The advantage of using steel trusses for building is that they are stronger than wood and greater open space inside a building is possible. They are ideal for barns, large storage buildings and commercial construction.

Fig. 3.1 A 3D truss model showing internal components like arrangement of truss and connection between bays
3.2. Analysis of Roof truss

Generally truss members are assumed to be joined together so as to transfer only the axial forces and not moments and shears from one member to the adjacent members (they are regarded as being pinned joints). The loads are assumed to be acting only at the nodes of the trusses. The trusses may be provided over a single span, simply supported over the two end supports, in which case they are usually statically determinate. Such trusses can be analysed manually by the method of joints or by the method of sections.

STAAD Pro is used for the analysis of truss. From the analysis based on pinned joint assumption, one obtains only the axial forces in the different members of the trusses. However, in actual design, the members of the trusses are joined together by more than one bolt or by welding, either directly or through larger size end gussets. Further, some of the members, particularly chord members, may be continuous over many nodes. Generally such joints enforce not only compatibility of translation but also compatibility of rotation of members meeting at the joint. As a result, the members of the trusses experience bending moment in addition to axial force. Further, the loads may be applied in between the nodes of the trusses, causing bending of the members. Such stresses are referred to as secondary stresses. The secondary bending stresses can be caused also by the eccentric connection of members at the joints. The analysis of trusses for the secondary moments and hence the secondary stresses can be carried out by an indeterminate structural analysis, usually using computer software which is negotiated in this paper.

The magnitude of the secondary stresses due to joint rigidity depends upon the stiffness of the joint and the stiffness of the members meeting at the joint. Normally the secondary stresses in roof trusses may be disregarded, if the slenderness ratio of the chord members is greater than 50 and that of the web members is greater than 100. The secondary stresses cannot be neglected when they are induced due to application of loads on members in between nodes and when the members are joined eccentrically.

3.3. Loads on the Roof truss

Dead load
Generally the dead load on the roof trusses in single storey industrial buildings consists of dead load of claddings and dead load of purlins, self weight of the trusses in addition to the weight of bracings etc. Further, additional special dead loads such as truss supported hoist dead loads; special ducting and ventilator weight etc. could contribute to roof truss dead loads. As the clear span length (column free span length) increases, the self weight of the moment resisting gable frames increases drastically. In such cases roof trusses are more economical. Dead loads of floor slabs can be considerably reduced by adopting composite slabs with profiled steel sheets

Live load
The live load on roof trusses consist of the gravitational load due to erection and servicing as well as dust load etc. and the intensity is taken as per IS:875-1975. Additional special live loads such as snow loads in very cold climates, crane live loads in trusses supporting monorails may have to be considered.
Wind load
Wind load on the roof trusses, unless the roof slope is too high, would be usually uplift force perpendicular to the roof, due to suction effect of the wind blowing over the roof. Hence the wind load on roof truss usually acts opposite to the gravity load, and its magnitude can be larger than gravity loads, causing reversal of forces in truss members.

Earthquake load
Since earthquake load on a building depends on the mass of the building, earthquake loads usually do not govern the design of light industrial steel buildings. Wind loads usually govern. However, in the case of industrial buildings with a large mass located at the roof or upper floors, the earthquake load may govern the design. These loads are calculated as per IS: 1893-2002.

3.4. Representative truss
A representative truss of span 52m is selected and designed as per the requirement. The shape of the roof truss is chosen to be arc of a circle for two reasons:

- The economy of an arched truss lies in the fact that the principal compression members follow approximately the line of greatest strain, so that the bracing can be made very light.
- The effect of wind loads on the truss can be reduced if we use a parabolic arch over pitched truss with low elevation.

A picture of the building assumed is presented in the Fig. 3.1.

FIG 3.2. REPRESENTATIVE FIGURE OF THE BUILDING
3.5. Various components of the building
In the chosen representative building, various components are,

- Truss members
- Purlins
- Beams connecting the trusses to form bays.
- Galvanized iron corrugated or profiled sheets
- Columns made up of RCC.
- External walls of 30cm thick brick.
CHAPTER 4

COMPUTER MODEL
4.1. Computer model

A model was selected on the basis of requirements of the consumer. A three-dimensional structural space truss model of the building is developed using computer aided drafting software (AutoCAD). The .dxf file of the truss is imported into the Analysis and Designing software (STAAD Pro) and Geometry is prepared which is followed by Analysis and Design.

The AutoCAD .dxf image files are shown in Fig. 4.1. and Fig. 4.2.

4.2. Coordinate system

The structure has been aligned $0^\circ$ to the True North. The principal axes in STAAD Pro are oriented as given below:

The X global axis is towards east
The Y global axis is pointing to upward.
The Z global axis is towards north
Fig. 4.2. 3D Elemental View of Entire Industrial Structure

Fig 4.3. STAAD Import of .dxf file
4.3. Members

Members and nodes in the STAAD Pro model are identified with member numbers nodal numbers. Member groups are created for better accuracy while assigning properties and load.

Fig. 4.4. Rendered Views after creating Geometry
CHAPTER 5

ANALYSIS CRITERIA

AND

ASSUMPTIONS
5. Analysis criteria and assumptions

Analysis procedure and criteria for the analysis are in accordance with design basis.

The roof truss and connections are designed in accordance with IS: 800-2007 general construction in steel-code of practice (third revision), the columns and footings are designed based on IS: 456-2000 plain and reinforced concrete- code of practice (fourth revision).

5.1. Analysis software

The structural analysis is performed using the Bentley’s STAAD Pro software which is based on finite element analysis technique, it also includes the modules for generation and application of earthquake loads, subsequent code checking of structural elements and joints and capabilities to carryout static analysis.

5.2. System of units

All engineering computations shall be presented in the system of international (SI) units as shown below:

- Length - Metres (m)
- Force - kilo Newton (kN)
- Moment- kilo Newton metre (kN-m)
- Stress - MPa or N/mm$^2$

5.3. Steel material

The material properties of the structural steel members are as listed below.

- Young’s modulus, E - 310,000N/mm$^2$
- Shear Modulus, G - 80,000 N/mm$^2$
- Density - 7850 kg/m$^3$
- Poisson’s ratio - 0.3
- Coefficient of Thermal Expansion- 11.7 $\times$ 10$^{-6}$/°C

All the structural members used are in accordance with structural steel specifications available in Indian market through TATA Structura.

5.4. Stress Criteria

Structural design stresses shall not exceed the allowable stresses as defined by IS 800 in accordance with the relevant steel grade and element profile and dimensions

5.5. Contingecies

For the structural analysis and verification, contingency factors considered are as follows
Table 5.1 contingency factors

<table>
<thead>
<tr>
<th>LOAD</th>
<th>CONTINGENCY FACTORS FOR ENGINEERING</th>
</tr>
</thead>
<tbody>
<tr>
<td>MAIN STRUCTURE</td>
<td>10%</td>
</tr>
<tr>
<td>SECONDARY STRUCTURE</td>
<td>10%</td>
</tr>
</tbody>
</table>
CHAPTER 6
ENVIRONMENTAL CONDITIONS
6. Environmental conditions

6.1. Wind loads
The following wind speeds are considered for analysis. One hour mean wind speed is used for the analysis.

<table>
<thead>
<tr>
<th>DIRECTION</th>
<th>WIND (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FOR ALL DIRECTIONS</td>
<td>44</td>
</tr>
</tbody>
</table>

6.2. Level of water table
It was assumed that water table is well below the foundation and does not affect the foundation.

6.3. Type of Foundation Soil
The soil is assumed to be hard rocky soil

6.4. Location and surroundings

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>MIYAPUR, HYDERABAD</th>
</tr>
</thead>
<tbody>
<tr>
<td>SURROUNDINGS</td>
<td>PLAIN AREA WITH BUSHES LESS THAN 0.5m HT.</td>
</tr>
</tbody>
</table>
CHAPTER 7
LOADS
AND
COMBINATIONS
7.1. Loads

**Structural loads or actions** are forces, deformations or accelerations applied to a structure or its components.

Loads cause stresses, deformations and displacements in structures. Assessment of their effects is carried out by the methods of structural analysis. Excess load or overloading may cause structural failure, and hence such possibility should be either considered in the design or strictly controlled. Engineers often evaluate structural loads based upon published regulations, contracts, or specifications. Accepted technical standards are used for acceptance testing and inspection.

Building codes require that structures be designed and built to safely resist all actions that they are likely to face during their service life, while remaining fit for use. Minimum loads or actions are specified in these building codes for types of structures, geographic locations, usage and materials of construction.

Structural loads are split into categories by their originating cause. Of course, in terms of the actual load on a structure, there is no difference between dead or live loading, but the split occurs for use in safety calculations or ease of analysis on complex models as follows.

To meet the requirement that design strength be higher than maximum loads, Building codes prescribe that, for structural design, loads are increased by **load factors**. These factors are, roughly, a ratio of the theoretical design strength to the maximum load expected in service. They are developed to help achieve the desired level of reliability of a structure based on probabilistic studies that take into account the load's originating cause, recurrence, distribution, and static or dynamic nature.

### 7.1.1. Types of loads

#### 7.1.1.1. Dead loads

The dead load includes loads that are relatively constant over time, including the weight of the structure itself, and immovable fixtures such as walls, plasterboard or carpet. Dead loads are also known as Permanent loads.

The designer can also be relatively sure of the magnitude of dead loads as they are closely linked to density and quantity of the construction materials. These have a low variance, and the designer himself is normally responsible for the specifications of these components.

![Fig.7.1. Direction of Dead load](image-url)
Dead loads are permanent loads that do not change in the structure’s life. They are,

- Self-weight of the structure
- Material incorporated into the structure: walls, floors, roofs, ceilings and permanent constructions
- Permanent equipments: fixtures, fittings, electrical wiring, plumbing tubes, ducted air system.
- Partitions, fixed and movable
- Stored materials

When there is significant design change, dead loads should be reassessed and followed by a fresh structural analysis.

Calculation of Dead loads is done as follows:

**Dead load of component = unit weight of the component x volume of the component**

Unit weight of various components are calculated from *IS: 875 (part1)*.

### 7.1.1.1. Movable partitions and removable items

**Movable partitions:**
Self weight of the movable partitions is the calculated actual weight but not less than a UDL of 0.5 kPa over the area being considered.

**Removable items:**
Consideration shall be given to the effect of removing permanent items that are not essential parts of the structure, such as water tanks, stored materials, service equipments, partitions and similar.

### 7.1.1.2. Load factors for dead loads

As the variability of dead loads in structure’s life is minimum, there are no factors to account for variability of dead loads. Dead loads are only factored to generate factored load combinations when design members for various limit states.

### 7.1.2. Live loads

Live loads are the result of the occupancy of a structure. In other words, it varies with how the building is to be used. For example, a storage room is much more likely to larger loads than is a residential bedroom. Bleachers at a stadium are likely to see larger loads than what is seen on a pitched building roof.

The specified live loads are generally expressed either as uniformly distributed area loads or point loads applied over small areas.
7.1.1.2.1. Uniformly distributed loads
The uniformly distributed loads are applied to portions of the structure that is likely to see a fairly uniform distribution of items over large areas (areas the size of a single room or larger). Where the live load $Q$ varies from one span (or room) to another, to account for the most adverse load cases, analysis is carried out for
- Factored live load on all spans
- Factored live load on two adjacent spans
- Factored live load on alternate spans

The uniformly distributed loads are calculated over the slabs, used for storage of material for floors, is calculated from Table 1 of IS: 875(part 2) as $2.4 \ kN/m^2$ for each metre of storage height.

**Total imposed load on the floor of building = 2.4 x height of storage**

7.1.1.2.2. Concentrated loads
Certain occupancies, such as office space, have the potential for a larger concentrated load (such as a large copy machine) being located in a space. This space may also be designed for uniformly distributed loads, but it is not probable that both the uniformly distributed load and the large concentrated load will occupy the space at the same time. Consequently the space must be designed to accommodate, separately, the uniformly distributed load and the point load, with the point load being moved around the space so as to cause maximum effect on the supporting elements.

A concentrated load shall be applied as follows:
(a) At its known position or where its position is not known, in the position giving the most adverse effect.
(b) Distributed over the actual area of application or if the actual area is not known.

7.1.1.2.3. Roof and supporting elements
Roofs are considered non-accessible except for normal maintenance and minor repairs. If roofs are frequently accessible and used for floor type activities are treated as floors. Roof load calculated according to Table 2 of IS 875 (part 2) for imposed loads as,

$$IL \ on \ roof = 0.75 - 0.52\gamma^2$$
\[
\frac{y}{h} = I
\]

\(h\) = the height of the highest part of the structure measured from its springing

\(I\) = Chord width of roof if singly curved or shorter of two chords if doubly curved

7.1.1.3. Wind loads

The wind pressure on a structure depends on the location of the structure, height of structure above the ground level and also on the shape of the structure. The code gives the basic wind pressure for the structures in various parts of the country. Both the wind pressures viz. including wind of short duration and excluding wind of short duration, have been given. All structures should be designed for the short duration wind. For buildings upto 10m in height, the intensity of wind pressure, as specified in the code, may be reduced by 25% for stability calculations and for the design of framework as well as cladding. For buildings over 10m and upto 30m height, this reduction can be made for stability calculations and for design of columns only. The total pressure on the walls or roof of an industrial building will depend on the external wind pressure and also on internal wind pressure. The internal wind pressure depends on the permeability of the buildings. For buildings having a small degree of permeability, the internal air pressure may be neglected. In the case of buildings with normal permeability the internal pressure can be ± 0.2p. Here ‘+’ indicates pressure and ‘-’ suction, ‘p’ is the basic wind pressure. If a building has openings larger than 20% of the wall area, the internal air pressure will be ±0.5 p.

Wind pressure on walls

The wind pressure per unit area ‘p’ on the wall is taken as 0.5p pressure on windward surface and 0.5p suction on leeward surface. When the walls form an enclosure, the windward wall will be subjected to a pressure of 0.5p and leeward wall to a suction of 0.5p. The total pressure on the walls will depend on the internal air pressure also.

For buildings with small permeability, design pressure on wall = 0.5p
For buildings with normal permeability, design pressure on wall = 0.7p
For buildings with large openings, design pressure on wall = p

Wind loads on roofs

The pressure normal to the slope of the roof is obtained by multiplying the basic pressure p by the factors given in Table 7.1. The table also shows the effect of internal pressure produced due to the permeability of the cladding or opening in walls and roof. If the wind blows parallel to the ridge of the roof, the average external wind pressure of the roof may be taken as -0.6p on both slopes of the roof over a length from the gable end equal to the mean height of the roof above the surrounding ground level and as -0.4p over the remaining length of the roof on both slopes.

When the wind blows parallel to a surface, a wind force acts on the surface in the direction of the wind. This force is called the ‘Wind Drag’. In the case of industrial buildings, when the wind blows
normal to the ridges, the wind drag is equal to 0.05p measured on plan area of roof and when the
direction of wind parallel to the ridge, wind drag is equal to 0.025p measured on plan area of roof.

Table 7.1. Wind loads on roofs

<table>
<thead>
<tr>
<th>Roof of</th>
<th>Zero Permeability</th>
<th>Normal Permeability</th>
<th>Large openings</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>External Pressure</td>
<td>$p_1 = +0.2p$</td>
<td>$p_1 = -0.2p$</td>
</tr>
<tr>
<td></td>
<td>Windward</td>
<td>Leeward</td>
<td>Windward</td>
</tr>
<tr>
<td>1</td>
<td>-1.00</td>
<td>-0.50</td>
<td>-1.2</td>
</tr>
<tr>
<td>10</td>
<td>-0.70</td>
<td>-0.50</td>
<td>-0.9</td>
</tr>
<tr>
<td>20</td>
<td>-0.40</td>
<td>-0.50</td>
<td>-0.6</td>
</tr>
<tr>
<td>30</td>
<td>-0.10</td>
<td>-0.50</td>
<td>-0.3</td>
</tr>
<tr>
<td>40</td>
<td>+0.10</td>
<td>-0.50</td>
<td>-0.1</td>
</tr>
<tr>
<td>50</td>
<td>+0.30</td>
<td>-0.50</td>
<td>+0.1</td>
</tr>
<tr>
<td>60</td>
<td>+0.40</td>
<td>-0.50</td>
<td>+0.2</td>
</tr>
<tr>
<td>70</td>
<td>+0.50</td>
<td>-0.50</td>
<td>+0.3</td>
</tr>
<tr>
<td>80</td>
<td>+0.50</td>
<td>-0.50</td>
<td>+0.3</td>
</tr>
<tr>
<td>90</td>
<td>+0.50</td>
<td>-0.50</td>
<td>+0.3</td>
</tr>
</tbody>
</table>

$p_1$ = internal pressure

In the multispan roofs with spans, heights and slopes nearly equal, the windward truss gives shelter to the other trusses. For general stability calculations and for the design columns, the windward slope of wind-ward span and leeward slope of leeward span are subjected to the full normal pressure of suction as given in table 7.1. and on all other roof slopes, only wind drag is considered (see Fig. 7.3.). For the design of roof trusses, however, full normal pressure or suction is considered on both faces, presuming that there was only one span. The wind pressures given above are the average pressures on a roof slope. For designing the roof sheeting or the fastenings of roof sheeting, we may take a larger wind pressure because these pressures may considerably exceed the average value on small areas. For designing roof sheeting and its fastenings, the values given in Table 7.1 may be increased numerically by 0.3p. In a distance equal to 15% of the length of the roof from the gable ends, fastenings should be capable of resisting a section of 2.0p on the area of the roof sheeting them support.

![Fig. 7.3. Wind drag](image)
The wind loads are calculated using **IS: 875(part3)** as

\[
\text{Wind Pressure} = 0.6 \times \frac{V_e^2}{g_1848}
\]

Where \(V_e\) = Design wind speed

\[
V_e = k_1 k_2 k_3 V_b
\]

- \(k_1\) = probability factor
- \(k_2\) = Terrain and height factor
- \(k_3\) = Topography factor

### 7.1.1.3. Seismic loads

Single storey industrial buildings are usually governed by wind loads rather than earthquake loads. This is because their roofs and walls are light in weight and often pitched or sloping and also because the buildings are permeable to wind which results in uplift of the roof. However, it is always safe to check any building for both wind and earthquakes.

Earthquake loading is different from wind loading in several respects and so earthquake design is also quite different from design for wind and other gravity loads. Severe earthquakes impose very high loads and so the usual practice is to ensure elastic behaviour under moderate earthquake and provide ductility to cater for severe earthquakes. Steel is inherently ductile and so only the calculation of loads due to moderate earthquake is considered. This can be done as per the IS 1893 code.

According to this code, a horizontal seismic coefficient times the weight of the structure should be applied as equivalent static earthquake load and the structure should be checked for safety under this load in combination with other loads as specified in IS 800. The combinations are as follows:

1. 1.5 \((DL + IL)\)
2. 1.2 \((DL + IL + EL)\)
3. 1.5 \((DL + EL)\)
4. 0.9 \(DL + 1.5 EL\)

The horizontal seismic coefficient \(A_h\) takes into account the location of the structure by means of a zone factor \(Z\), the importance of the structure by means of a factor \(I\) and the ductility by means of a factor \(R\). It also considers the flexibility of the structure foundation system by means of an
acceleration ratio \(\frac{S_a}{g}\), which is a function of the natural time period \(T\). This last ratio is given in the form of a graph known as the response spectrum. The horizontal seismic coefficient \(A_h\) is given by

\[
A_n = \frac{Z I S_a}{2 R g}
\]

Where,

- \(Z\) = Zone factor corresponding to the seismic zone obtained from a map,
- \(I\) = Importance factor,
- \(R\) = Response reduction factor,
- \(\frac{S_a}{g}\) = Spectral Acceleration Coefficient

<table>
<thead>
<tr>
<th>Seismic Zone</th>
</tr>
</thead>
<tbody>
<tr>
<td>II</td>
</tr>
<tr>
<td>III</td>
</tr>
<tr>
<td>IV</td>
</tr>
<tr>
<td>V</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Seismic Intensity</th>
<th>II</th>
<th>III</th>
<th>IV</th>
<th>V</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>0.10</td>
<td>0.16</td>
<td>0.24</td>
<td>0.36</td>
</tr>
<tr>
<td>Moderate</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Severe</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Very Severe</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

For industries using hazardous materials and fragile products the importance factor may be taken as 1.5 but for most industries it may be taken as 1.0. The Response reduction factor \(R\) may be taken as 4 for buildings where special detailing as per section 12 of IS 800 has not been followed.

The natural time period \(T\) is very important and should be calculated correctly. For single storey structures, it may be taken as \(T = 2\pi \sqrt{\frac{k}{m}}\) where \(k\) is the lateral (horizontal) stiffness of the supporting structure and \(m\) is the mass of the roof usually taken as the sum of the roof dead load plus 50% of the live load divided by the acceleration due to gravity \(g\). Guidelines for calculating \(k\) in some simple cases are given in Fig. 7.5.

Finally, the acceleration ratio \(\frac{S_a}{g}\) can be obtained from the graph corresponding to the soil type as shown in Fig. 7.6. In this figure, medium soil corresponds to stiff clay or sand and soft soil corresponds to loose clay and loamy soils.
7.1.2. Load Case Details

<table>
<thead>
<tr>
<th>Load Case No</th>
<th>Load Case name</th>
<th>Load value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>EQX</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>EQ-X</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>EQZ</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>EQ-Z</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>DL</td>
<td>0.13kN/m</td>
</tr>
<tr>
<td>6</td>
<td>LL</td>
<td>0.74kN/m</td>
</tr>
<tr>
<td>7</td>
<td>WLX</td>
<td>1.47 kN/m²</td>
</tr>
<tr>
<td>8</td>
<td>WL-X</td>
<td>1.47 kN/m²</td>
</tr>
<tr>
<td>9</td>
<td>WLZ</td>
<td>1.47 kN/m²</td>
</tr>
<tr>
<td>10</td>
<td>WL-Z</td>
<td>1.47 kN/m²</td>
</tr>
</tbody>
</table>

7.2. Load combinations

The design load combinations are the various combinations of possible load cases for which the structure needs to be designed. For IS 456-2000, if a structure is subjected to dead (D), live (L), pattern live (PL), snow (S), wind (W), and earthquake (E) loads, and considering that wind and earthquake forces are reversible, the following load combinations may need to be considered (IS 36.4, Table 18):
Of the above Load Combinations, only the possible combinations are considered in the design of this building. They are listed in the following table:

Table 7.4. Various combinations of loads considered

<table>
<thead>
<tr>
<th>LOAD COMBINATION NUMBER</th>
<th>LOAD COMBINATION NAME</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>1.5(DL+LL)</td>
</tr>
<tr>
<td>8</td>
<td>1.2(DL+LL+EQX)</td>
</tr>
<tr>
<td>9</td>
<td>1.2(DL+LL+EQ-X)</td>
</tr>
<tr>
<td>10</td>
<td>1.2(DL+LL+EQ-Z)</td>
</tr>
<tr>
<td>11</td>
<td>1.2(DL+LL+EQ-Z)</td>
</tr>
<tr>
<td>12</td>
<td>0.9DL+1.5EQX</td>
</tr>
<tr>
<td>13</td>
<td>0.9DL+1.5EQ-X</td>
</tr>
<tr>
<td>14</td>
<td>0.9DL+1.5EQZ</td>
</tr>
<tr>
<td>15</td>
<td>0.9DL+1.5EQ-Z</td>
</tr>
<tr>
<td>16</td>
<td>1.5(DL+WLX)</td>
</tr>
<tr>
<td>19</td>
<td>1.5(DL+WLZ)</td>
</tr>
</tbody>
</table>
CHAPTER 8

ANALYSIS, DESIGN AND RESULTS
8.1. ANALYSIS AND RESULTS

The results of the STAAD analysis are provided in appendix 2 at the end.

Results include,

- Analysis and design of steel truss members
- Analysis of purlins
- Design of RCC columns up to plinth level
- Support reactions at column bases at plinth level.

The brief summary of the analysis is provided in the following points
CHAPTER 9

DESIGN
OF
PLINTH FLOOR
9. DESIGN OF FLOOR SLAB:

9.1. TYPE OF SLAB USED:
The slabs to be designed are of size 50m x 5 m. Since, $\frac{L_y}{L_x} > 2$ the slab is to be designed as ONE-WAY SLAB.

9.2. Design of One-way Slabs

The procedure of the design of one-way slab is the same as that of beams. However, the amounts of reinforcing bars are for one metre width of the slab as to be determined from either the governing design moments (positive or negative) or from the requirement of minimum reinforcement. The different steps of the design are explained below.

**Step 1: Selection of preliminary depth of slab**
The depth of the slab shall be assumed from the span to effective depth ratios.

**Step 2: Design loads, bending moments and shear forces**
The total factored (design) loads are to be determined adding the estimated dead load of the slab, load of the floor finish, given or assumed live loads etc. after multiplying each of them with the respective partial safety factors. Thereafter, the design positive and negative bending moments and shear forces are to be determined using the respective coefficients given in Tables 12 and 13 of IS 456.

**Step 3: Determination/checking of the effective and total depths of slabs**
The effective depth of the slab shall be determined employing

$$M_{u,\text{lim}} = R_{\text{lim}} \frac{bd^2}{2}$$

Where the values of $R_{\text{lim}}$ for three different grades of concrete and three different grades of steel are given in IS 456-2000. The value of $b$ shall be taken as one meter.

The total depth of slab shall then be determined adding appropriate nominal cover (table 16 and 16a of clause 24.6 of is: 456-2000) and half of the diameter of larger bar if different size bars are provided. Normally, the calculated depth comes out to be much less then assumed depth. However the final selection of depth of slab must be done only after checking for shear.

**Step 4: Depth of the slab for shear force**
Theoretically, the depth of the slab can be checked for shear force if the design shear strength of concrete is known. Since this depends upon the percentage of tensile reinforcement, the design shear strength shall be assumed considering the lowest percentage of steel. The value of $c\tau$ shall be modified after knowing the multiplying factor $k$ from the depth tentatively selected for the slab in Step 3. If necessary, the depth of the slab shall be modified.

**Step 5: Determination of areas of steel**
Area of steel reinforcement along the direction of one-way slab should be determined employing Eq
$$M_u = 0.87 \, f_y \, A_{st} \, d \left\{ 1 - \left( A_{st} \right) \left( f_y \right) / \left( f_{ck} \right) \left( b, d \right) \right\}$$

The above equation is applicable as the slab in most of the cases is under-reinforced due to the selection of depth larger than the computed value in Step 3. The area of steel so determined should be checked whether it is at least the minimum area of steel as mentioned in cl.26.5.2.1 of IS 456.

Alternatively, tables and charts of SP-16 may be used to determine the depth of the slab and the corresponding area of steel. Tables 5 to 44 of SP-16 covering a wide range of grades of concrete and Chart 90 shall be used for determining the depth and reinforcement of slabs. Tables of SP-16 take into consideration of maximum diameter of bars not exceeding one-eighth the depth of the slab. Zeros at the top right hand corner of these tables indicate the region where the percentage of reinforcement would exceed $p_{t,\lim}$. Similarly, zeros at the lower left and corner indicate the region where the reinforcement is less than the minimum stipulated in the code. Therefore, no separate checking is needed for the allowable maximum diameter of the bars or the computed area of steel exceeding the minimum area of steel while using tables and charts of SP-16.

The amount of steel reinforcement along the large span shall be the minimum amount of steel as per cl.26.5.2.1 of IS 456.

**Step 6: Selection of diameters and spacings of reinforcing bars**

(cl.26.5.2.2 and 26.3.3 of IS 456)

The diameter and spacing of bars are to be determined as per cl.26.5.2.2 and 26.3.3 of IS 456. As mentioned in Step 5, this step may be avoided when using the tables and CHARTS OF SP-16.

Fig11.1. Figure showing reinforcement details of the slab.
CHAPTER 10
DESIGN
OF
FOOTINGS
10. DESIGN OF FOOTINGS

10.1. BASIS OF DESIGN OF FOOTINGS

Footings under walls are called one way footings and those under columns, two way footings. The first step in design is to calculate the necessary area from the formula

\[
\text{Area of footings} = \frac{\text{service load on column or wall above}}{\text{safe bearing capacity of soil}}
\]

Having thus determined the size of the footings, its structural design is carried out by using factored loads and principles of limit state design already discussed in the case of other R.C members. The main items to be designed are the thickness of the footing and its reinforcement. The thickness should be sufficient to

- Resist shear force without shear steel and the bending moment without compression steel
- Give the structure the required structural rigidity so that foundation reaction below can be assumed (see sections 22.4 and 22.5)
- Withstand the corrosion that can be caused from the ground. (This minimum cover is required not less than 40mm when it is cast against a layer of building concrete of 75 to 80mm thickness.)

It is also important to remember that the percentage of steel provided should not be less than 0.15 for Fe 250 and 0.12 for Fe 415 steels as specified for slabs in IS 456; clause 22.5.2.1

**Design Principles:**

- Footings shall be designed to sustain the applied loads, moments and forces and safe bearing capacity is not to exceeded, vide clause 34.1
- In R.C.C. footing, the thickness at the edge shall not be less than 150 mm for footings on soils. Vide clause 34.12
- The greater B.M to be used in the design of an isolated concrete footing which support a column, pedestal or wall shall be computed in the manner prescribed in clause 34.2.3.1(a)
- The critical section for diagonal cracking is taken at a distance equal to effective depth from the face of the column in hard soil vide clause 34.2.4.3(a) and shall not exceed nominal shear stress.

10.2. Design of Footing for the Design Load Applied on the Column

**Type of footing:**

- Rectangular footing
- Size of column on the footing = 400 X 650 mm

**Loads on the footing:**
Load on footing $P_u$ = 500 KN
Moment arrived from analysis $M_u$ = 1.418 KN m
Factored moment $1.5 \times 1.714 = 2.127$ KN m
Axial load on the column = 500 KN
Weight of foundation at 10% of column load = 50KN
Total load bearing capacity of soil = 300 kN/m²

Size of the footing:

Area of the footing = 600/300 = 2 m²

The dimensions of footing in plan must be such that the projections of beyond the column faces are equal.

Let
- The length of the footing be $L$,
- The width of the footing be $B$, $a = 650/400 = 1.625$

hence, $1.625B = 2m^2$

hence, $B = 1.23$ m say 1.25m

$L = 2$ m

Provide size of footing = 2 m x 1.25 m

Stress on the footing $= \frac{P}{B} \pm \frac{My}{I}$

$I = \frac{1}{12}x2x1.25^3$

$= 0.33 m^4$

$y = 1.5/2 = 0.75 m$

stress on the footing $= \frac{550}{125} \pm \frac{2.127 \times 0.75}{0.33}$

$= 275 \pm 4.834$

$P_{max} = 275 + 4.834 = 279.834$ kN/m² < 300 kN/m²

$P_{min} = 275 - 4.834 = 271.046$ kN/m² < 300 kN/m²

Hence design footing can bare a maximum pressure of 279.834 kN/m²

Projections beyond column faces $= \frac{2 - 0.75}{2}$

$= 0.625$
And 
\[ \frac{1.5 - 0.38}{2} = 0.56 \text{ m} \]

Net upward on the foundation 
\[ = \frac{600}{(2 \times 1.5)} \]
\[ = 200 \text{ kN/m}^2 \]

Hence the footing design for maximum pressure of 200 kN/m²

**Bending moment along x-x axis:**

\[ M_{x-x} = \frac{P(L-a)^2}{8L} = \frac{600(2-0.75)}{8 \times 1.5} \]

Eff. Depth \( d \) = 62.5mm

Hence provide an effective depth of footing 150mm

Assuming diameter of bar is 20 mm \( \phi \) with clear cover of 50 mm

Overall depth = 150+60 = 210mm

**Area of reinforcement:**

**Steel required to long direction for B.M \( M_x \)**

\[ M_u = 0.87f_y \times 415 \times A_{st} \times [210 - (0.42 \times 0.48 \times 210)] \]
\[ A_{st} = 2514 \text{ mm}^2 \]

Provide 8 No.s of 20 mm \( \phi \) bars at a spacing of 90mm c/c distance.

Hence provide area of steel =2514 mm²

**Steel required in short span direction:**

\[ M_u = 0.87 f_y A_{st} (d-0.42x_{\text{umax}}) \]
\[ 212.8 \times 10^6 = 0.87 \times 415 \times A_{st} (210-0.42 \times 0.48 \times 210) \]

Hence \( A_{st} = 1200 \text{ mm}^2 \)

Provide No.s of 20 mm dia bars at a spacing of 190mm c/c distance.

Hence provide area of steel in central band of 1800mm

\[ \frac{2}{\beta + 1} \times A = \frac{2}{1.8/3.6 + 1} \times 1642 \]
\[ = 2190.1 \text{ mm}^2 \]
Provide 7 No.s of 20 mm dia bars at a spacing of 140 mm c/c distance, and provide one more bar of 20 mm diameter at each end.

**Check for two way shear**

As per IS 456-2000, the critical section for two way shear at a distance of \( \frac{d}{2} \) from the face of the column.

Taking a section at \( \frac{d}{2} \) around column, we get

\[
V = P \times [A-(a+d) \times (b+d)]
\]

\[
= 284 \times [2-(0.65+0.15) \times (0.4+0.15)]
\]

\[
= 443 \text{ KN}.
\]

Nominal shear stress \( \tau_v = \frac{V}{bd} \)

\[
= 1391.4*1000(2*(300+540+750+540)*540)
\]

\[
= 0.58 \text{ N/mm}^2
\]

Shear strength of \( M_{20} \) concrete

\[
\tau'_c = k_s \tau_c
\]

\[
k_s = (0.5 + \beta_c)
\]

\[
\beta_c = \frac{\text{length of shorter side of column}}{\text{length of longer side of column}}
\]

\[
= \frac{380}{750} = 0.506
\]

\[
K_s = 0.5 + 0.506
\]

\[
= 1.006 < 1.000
\]

\[
K_s = 1.0
\]

\[
\tau'_c = \tau_c = 0.25(f_{ck})^{0.5}
\]

\[
= 1.12 \text{ N/mm}^2
\]

Hence safe.

**Check for one way shear**

For maximum \( V \), take section along the breadth in the y-y direction at a distance ‘\( d \)’ from the column.
\[ V_{\text{max}} = \frac{p \cdot B (L - a - 2d)}{2} \]
\[ = 284 \times 1.8 \times (3.6 - 0.75 - 2 \times 0.54)/2 \]
\[ = 503.53 \text{ kN} \]

Nominal shear stress \( \tau_p = \frac{V_u}{bd} \)
\[ = \frac{503.53 \times 1000}{1800 \times 540} \]
\[ = 0.46 \text{ N/mm}^2 \]

Allowable shear stress, can be calculated by percentage of steel.
\[ P = \frac{100 + 3455.8}{1800 \times 540} = 0.4\% \]
For \( P = 0.4\% \)
\[ \tau_c = 0.47 \text{ N/mm}^2 \]
Therefore \( \tau_p < \tau_c \)
Hence safe.
CHAPTER 11

DETAILS OF CONNECTIONS
11.1. JOINT DESIGN FOR WELDED TUBULAR STEEL STRUCTURES

11.1.1. Application and nomenclature

The following procedures are concerned with the static design of tubular joints formed by the full penetration welding of two or more tubular members. The procedures apply to tubular joints fabricated from steel plate satisfying the requirements of BS 4360(4) or from seamless tubular to equivalent specifications.

The word 'simple' in the following procedures refers to joints without overlap of brace members and without the use of gussets, diaphragms, stiffeners or grout. Overlapping joints are defined as joints in which the brace forces are partially transferred in shear between overlapping braces through their common weld.

The design procedures are developed from consideration of the characteristic strength of tubular joints.

Characteristic strength is defined as that value below which not more than 5% of the results of an infinite number of tests would fall.

The notation used within this section is as follows:

- \( d \) Outside diameter of brace
- \( D \) Outside diameter of chord at brace intersection
- \( e \) Joint eccentricity
- \( F_y \) Characteristic yield stress of chord member
- \( g \) Gap between braces for K and YT joints
- \( K_a \) Relative length factor
- \( M \) Moment load
- \( M_{ci}, M_{co} \) Permissible in-plane and out-of-plane moment strengths
- \( M_{di}, M_{do} \) Design acting in-plane and out-of-plane moment loads
- \( M_i, M_o \) n-plane and out-of-plane moments
- \( M_{ki}, M_{ko} \) Characteristic in-plane and out-of-plane moment strengths
- \( P, P_1, P_2, P_3 \) Axial load
- \( P_c \) Permissible axial strength
- \( P_d \) Design acting axial load
- \( P_k \) Characteristic axial strength
- \( Q_f \) Chord load factor
- \( Q_g \) Coefficient for K joints
- \( Q_u \) Strength factor for various joint and load types
- \( Q'\$ \) Geometrical modifier
- \( t, t_1, t_2 \) Wall thickness of brace
- \( T \) Wall thickness of chord
- \( U \) Chord utilization
11.1.2. Determination of design loads

Applied loads should be calculated by established structural analysis methods taking account of appropriate environmental and dynamic conditions. Considerations should be given to the effect on accuracy of the following:

- Variations in wall thickness and outside diameter of members
- Representation of joint cans
- Representation of brace stubs
- Joint eccentricities
- Working point offsets

Large joint eccentricities should be modelled in the analysis at design stage. In some instances it may be appropriate to consider the effects of smaller eccentricities and/or the effects of local joint flexibilities on member and joint loads. Requirements for modelling of joint eccentricities and procedures for the inclusion of joint eccentricity effects on local joint flexibilities need careful consideration.

In overlapping joints there is a transfer of load between brace members through their common weld and it is essential to recognise these transfer components to avoid incorrect and possible unsafe estimation of the total load resisted by the chord. These transfer components are dependent on the joint geometry and configuration and on the type and magnitude of incoming brace loads. Accordingly, each overlapping joint should be treated on an individual basis. Design acting loads for each load component should be taken as:

$$P_d, M_{di}, M_{do} = \text{calculated applied load}$$

11.1.3. Classification of joints for static strength design

Each joint should be considered as a number of independent chord/brace intersections and the capacity of each intersection should be checked against the design requirements set out in paragraph below. Each plane of a multiplanar joint should be subjected to separate consideration and classification. Each chord/brace intersection should be classified as Y, K or X according to their configuration and load pattern for each load case. Examples of joint classification are shown in Figure 2 and should be used with the following guidelines:

- For two or three brace members on one side of the chord, the classification is dependent on the equilibrium of the axial load component in the brace members. If the resultant shear on the chord member is essentially zero, the joint should be allocated a K classification. If this requirement is not met, the joint can be downgraded to Y classification as shown in Figure 2. However, for braces which carry part of their load as K joints and part as Y or X joints, interpolation based on the portion of each in total may be valid.
- For multibrace joints with braces on either side of the chord care should be taken in allocating the appropriate classification. For example, a K classification would be valid if the net shear across the chord is essentially zero. In contrast, if the loads in all the braces are
tensile (e.g. at a skirt pile connection), even an X classification may be unsafe due to the increased ovalising effect.

11.1.4. Factors affecting the strength of a tubular joint

The following principal factors have been shown to affect the strength of a given simple tubular joint:

- Chord outside diameter (D)
- Brace outside diameter (d)
- Chord wall thickness (T)
- The included angle between chord and brace (q)
- Gap between braces (for K joints only) (g)
- Chord material yield stress (Fy)

The static strength of a joint may be enhanced by the presence of ring and longitudinal stiffeners, gussets or cementitious filling (e.g. grouting). Comprehensive design formulae are not available for all cases and special consideration should be given to such joints.
NOTES
1) The loads $P_1$, $P_2$ and $P_3$ are taken to act in the direction shown.
2) For all cases above check each brace separately.
3) This figure should be read in conjunction with Section 13.1.3, for further information.

\[ \alpha = 2L/D \]
\[ \beta = d/D \]
\[ \gamma = D/2L \]
\[ \zeta = g/D \]
\[ \tau = V/T \]

Figure 11.1. Nomenclature for example non-overlapping and overlapping joints
### 11.1.5. Characteristic strength of joints

The characteristic strength of a welded tubular joint subjected to unidirectional loading may be derived as follows:

Where,

\[ P_k = \text{characteristic strength for brace axial load} \]
\[ M_{ki} = \text{characteristic strength for brace in-plane moment load} \]
\[ M_{ko} = \text{characteristic strength for brace out-of-plane moment load} \]
\[ F_y = \text{characteristic yield stress of the chord member at the joint (or 0.7 times the characteristic tensile strength if less). If characteristic values are not available specified minimum values may be substituted.} \]
Ka = (1 + 1/sin2)/2

Qf is a factor to allow for the presence of axial and moment loads in the chord. Qf is defined as:
Qf = 1.0 - 1.638 lgU2 for extreme conditions Equation 2.2
= 1.0 - 2.890 lgU2 for operating conditions

Where,
8 = 0.030 for brace axial load
= 0.045 for brace in-plane moment load
= 0.021 for brace out-of-plane moment load

And,

\[ U = \frac{\sqrt{(0.23PD)^2 + M_t^2 + M_o^2}}{0.72D^2 T F_y} \]

with all forces in the function U relating to the calculated applied loads in the chord. Note that U defines the chord utilisation factor. Extreme and operating conditions are discussed in Offshore Technology Report chord axial tension force ≥ + Equation 2.4 with all forces relating to the calculated applied loads in the chord.

Qg is a strength factor which varies with the joint and load type. Qg is defined in Table 13.1.

### Table 11.1 Coefficient Qu

<table>
<thead>
<tr>
<th>Load direction</th>
<th>Coefficient Qu for various joint design classifications</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Y</td>
</tr>
<tr>
<td>Axial compression</td>
<td>(2 + 20β) √Q5s</td>
</tr>
<tr>
<td>Axial tension</td>
<td>(8 + 22β)</td>
</tr>
<tr>
<td>In-plane bending *</td>
<td>5Py/v_2sinθ</td>
</tr>
<tr>
<td>Out-of-plane bending</td>
<td>(1.6 + 7β) Qu.9</td>
</tr>
</tbody>
</table>

Qg = 1.7 - 0.92β² but should not be taken as less than 1.0

Qfβ is the geometrical modifier defined as follows:

Qfβ = 1.0 for β ≤ 0.6

\[ = \frac{0.3}{β(1 - 0.833β)} \text{ for } β ≥ 0.6 \]
11.2. Nut –bolt connections
This is another type of connection used to connect the members of structure. This type of connection is designed in the present project for connecting the truss to the concrete columns. These connections are designed according to IS800

Fig 11.3. Bolted Connections
CHAPTER 12

CONCLUSIONS
12.1. Conclusions:

The aim of this project, “to learn, practice and Excel in various subjects which we learned in our classrooms by applying them practically by analysis and design of an arched roof truss, for the usage of industrial storage of materials, efficiently to reach the requirement as well as economy”, has been fulfilled successfully.

The various operations of the building have been analyzed perfectly and the local codes have been followed correctly. The analyzed truss has strength to withstand various loads.

The manual design of other components are also given high importance and calculations of desired reinforcement is found with high factors of safety.

We propose that this building has adequate strength to resist all the loads and meet its purpose of storage of materials in its life span. STAAD analysis results show that the structure can resist various loads coming on to it.
CHAPTER 13

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15. REFERENCES

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