THESIS REPORT

ON

“INDUSTRIAL BUILDING DESIGN USING AUTODESK REVIT”

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INDUSTRIAL BUILDING DESIGN USING AUTODESK REVIT

Submitted in partial fulfillment of the requirements of the award of the degree of

Master of Technology

In

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by

UTKARSH

(**18032010339**)

Under the guidance of

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CERTIFICATE

This is to certify that the project work entitled **“Industrial Building Design Using Autodesk Revit**" submitted by **Utkarsh (18032010339)** to the School of Civil Engineering, Galgotias University, Greater Noida, for the award of the degree of **Master of Technology in Civil Engineering** is a bonafide work carried out by him/her under my supervision and guidance. The present work, in my opinion, has reached the requisite standard, fulfilling the requirements for the said degree.

The results contained in this report have not been submitted, in part or full, to any other university or institute for the award of any degree or diploma.

(Dr.Suprakash Biswas)	**(Prof. (Dr) Manju Dominic)**
Asst. Professor	**Dean,**
Guide	**School of Civil Engineering**

External Examiner

DECLARATION

I declare that this written submission represents my ideas in my own words and where others' ideas or words have been included, I have adequately cited and referenced the original sources. I also declare that I have adhered to all principles of academic honesty and integrity and have not misrepresented or fabricated or falsified any idea/data/fact/source in my submission. I understand that any violation of the above will be cause for disciplinary action by the Institute and can also evoke penal action from the sources which have thus not been properly cited or from whom proper permission has not been taken when needed.

Date:	**(Utkarsh)**
Place:	**18SOCE2010008**

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INTRODUCTION

1.1 PROBLEM DEFINATION

Study and construction of economical and stable roofing truss for industrial use, such as sheds, workshops, warehouses, etc., using AUTODESK REVIT and manual calculations.

1.2 SCOPE

The key focus of this project is to extend class room expertise in the real world through the architecture of a roofed house. Such buildings need wide and open spaces unobstructed by columns. The broad floor space offers ample stability and facility for subsequent improvements in the manufacturing configuration without significant alterations to the structure. Industrial buildings are built with an sufficient headroom for the use of an overhead moving crane.

1.3 GENERAL

Steel-framed houses are widely used for manufacturing purposes. They are divided into three different categories:

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

In the construction of industrial buildings, loading conditions and spatial considerations will determine the degree of complexity and, ultimately, the economy. The builder should have clear knowledge of the manufacturing process or function for which the building is planned. An optimal balance between protection, feature and economy can be accomplished in this way. The key dimensions of an industrial building are typically calculated by a mix of practical and architectural factors. Its breadth is determined first from the owner's analysis of the area needed for processing or storage operations. The model must then decide whether a single transparent span will accommodate this width effectively, or whether multi-bay span is feasible. Similarly, the customer usually easily selects the entire length, but the designer must consider the port's optimum length. Many of the considerations affecting the decision are: foundation conditions and their ability to accept the loads on the columns.

Again, building height needs to be regarded as practical, the crucial aspect is the clearance below the hook for buildings with overhead cranes. There is no snow in Hyderabad and thus fairly low roof pitches can be accomplished. The steeper the pitch the smoother the structural operation, however, increased cost of sheeting typically outweighs this benefit. In practice roof pitches are favoured between 5 and 10. Such pits are ideal for all of the sheet roof types in continuous ranges, some of which are appropriate for pitches up to 1.

1.4 BUILDING CONFIGURATION OF A COMMON INDUSTRIAL STRUCTURE

In addition, frames mask the position of width in commercial buildings. Various these frames are placed at the correct distance to achieve the required length. Many bays may be designed surrounding each other, depending on the necessity. Depends on the gap between the column line, the headroom or clearance necessary the quality of the roofing material and the form of

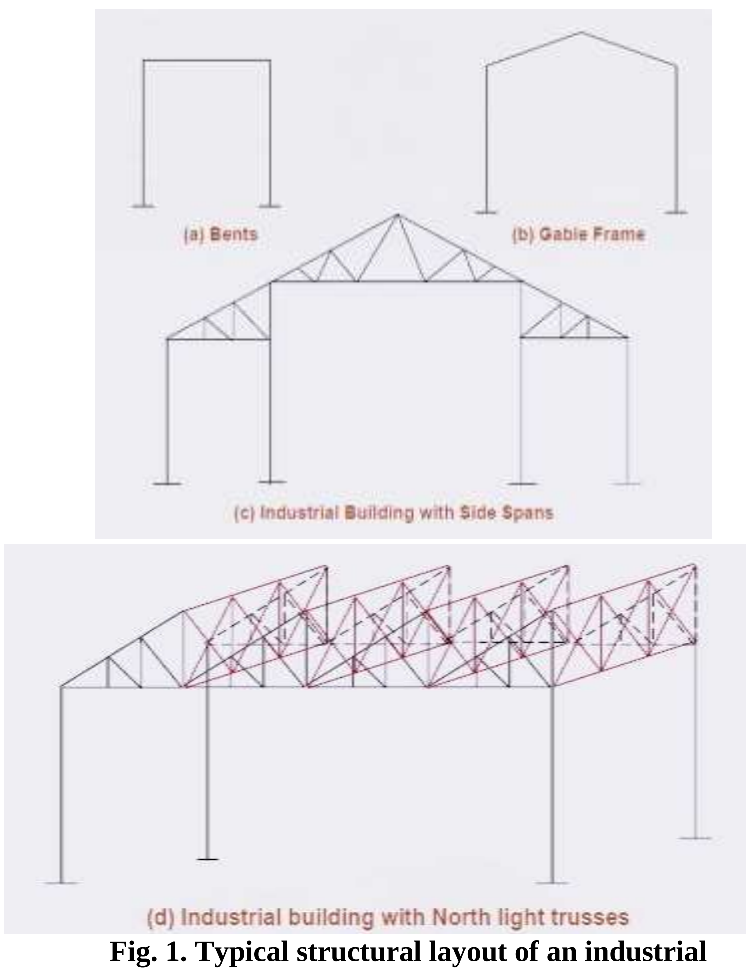


Fig. 1. Typical structural layout of an industrial

Lighting. If the span is smaller, portal frames such as stainless steel bends or gable frames may be used, although buildings with trusses may be used if the span is wide.

In single and multi-story structures, the horizontal and vertical bracings are often used to avoid primary wind and other lateral loads. Through crane boom in industrial buildings this breach minimizes the difference between the frames. The foundations of small and big buildings are often held horizontally, which increasing the buckling ability.

1.4.1 FLOORS

Different floor types, such as manufacturing, warehouses, storage, machinery, and management are required from every factory's usage. The standards of operation in these areas differ widely, such that different floor types are required. Depending on the form of operation performed, the industrial floors shall have ample abrasion resistance, effects, acid action and temperatures. High strength and high efficiency concretes can commercially fulfill each of these criteria and are the substance used most commonly. .The base of vibrating equipment should be mounted on rock or on solid soil (such as reciprocating and high-speed spinning machines) and should be isolated from the neighboring floor to avoid vibrations.

1.4.2 ROOF SYSTEM

The manufacturer will try consistency of lightweight, power, water resistance, insulation, fire protection, efficiency, reliability and low maintenance efficiency while constructing a roof. Blocks, purl and roof supports with column support have a can roofing structure for commercial buildings. They are structural. During the roof system construction, specific considerations are taken into consideration: form of roof cover, insulation benefit, acoustic properties, and visibility on the inner side, weight and maintenance. The roof-cover components are towed with galvanized iron corrugated sheets or profiled sheets such as Asbestos, Corrugated and Trafford cement sheets or ductile sheets. Depending the form of sheeting, the deflection mark for purlins and trusses. Small deflection values are defined in the code for delicate sheeting.

1.4.3 LIGHTING

When adequate illumination is provided, industrial operations can be performed most efficiently. The strength and uniformity of good illumination are necessary. Since natural light is abundant, daylight is cost-efficient and prudent to be used in manufacturing plants whenever feasible for lighting. Side windows are very useful to light the interior of small buildings, but in the case of large buildings they are not so successful. Monitors are suitable for large buildings (Fig. 2.).

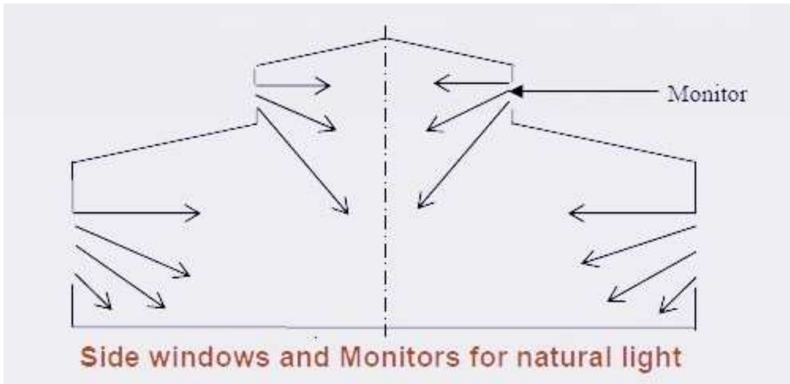


Fig. 2 Side windows and Monitors for natural light

1.4.4 VENTILATION

Also critical is ventilation of the industrial buildings. For heat absorption, dust reduction, cold and clean fresh air replacement ventilation is used. Ventilation is used. It can be achieved with natural forces such as aeration or with industrial machinery including suppliers.. Low-level openings for natural light and high-level outlets for ventilation are open to benefit on the elevated roof windows and display.

CHAPTER 2

REVIEW OF LITERATURE

2.1 REVIEW OF LITERATURE

1..Ms. Aayillia. And K. Jayasidhan says the multi-story commercial building architecture and study. The house is built in Koratty on a ground floor + 3 story. The research and configuration is carried out as far as possible in keeping with the required specification. The framework review was conducted using the STAAD PRO.V8i software kit. All structural elements were manually developed. In AutoCAD 2013, specifics of the strengthening were given. The implementation of the program saves time. Its worth better than physical jobs.

2. C. M. Meera talks of the PEB definition and CSB principle comparative research. The thesis is carried out utilizing the principles and interpretation of the built structures, using the structural analysis and software Staad. Pro, by modeling a standard structure of the planned Industrial Warehouse project.

3. Laxmikant Vairagade, Swapnil D. Bokade, discusses two styles of industrial buildings. The two buildings are traditional buildings and a modern idea of single story urban architecture is the term for Pre-engineered Construction (PEB). This approach is flexible not only because of its pre-design and pre-production consistency, but also because of its light weight and economic construction. The definition requires the methodology where optimal criteria are implemented to include the best segment possible. This design has a range of benefits over the CSB model of roofed buildings.

4. The research on the static and fluid analytics and architecture of pre-engineered buildings and traditional steel frames is proposed by Aslam Hutagi Aijaz Ahmad Zende, Prof. A. V. Kulkarni. In Staad Pro software, the framework is built and compared to the standard model, which in effect reduces the expense.

5. In the article of Sagar.D, the Industrial Steell trusts was contrasted to Pre-engineering buildings with columns of the same size: 14 m by 31,50 m, 20 m by 50 m, 28 m by 70 m and 5,25 m, 6,25 m and 7 m by column height respectively. IS 800-2007 (LSM) architecture is focused on the load of the dead load , live load and wind load as well as the variations defined in the IS. For base columns as reference hinge, research findings are observed. Tests of Commercial stainless steel systems are

contrasted with the same pre-engineering design proportions.

6. Yash Patel, Yashveersinh Chhasatia, Shreepalsinh Gohil, Het Parmar, says the steel structure is composed of orthhodox steel parts built and constructed through traditional approaches. This contributes to bulky systems or to inefficient ones. Tubular steel with its relatively improved performance is the highest potential alternative to the standard one. For certain institutional participants, death weight appears to be reduced, and it is obvious that the segment of the tube helps minimize the economy overall. Economy is the main objective of present research, including contrast for different conditions of traditional structures with tubular structure. Results show that the use of tubular parts saves up to 15 to 25% on cost.

7. Darshan Kalantri, Sujay Deshpande, the author of Pavan Gudi, says the construction of buildings has been made possible by the Preengineered Building (PEB). The implementation of the PEB model instead of the CSB concept resulted in certain benefits, as participants have been built in line with the schematic of the bending moment to minimize the material required. This approach is flexible not only because of its pre-design and pre-fabrication quality, but also because of its light weight and economic construction. This design provides some benefits over the CSB model of roof truss buildings.

8. This review of several publications concerning comparative studies of conventional and pre-engineered buildings is a paper of Pradip S. Lande, Vivek, V. Kucheriya. The key purpose of the book is to discuss the relevant theoretical and design analyses of modern steel production and pre-engineered production as well as the economic implications of cold-formed steel for hot-rolled purlins.

9. Laxmi R. Gupta, author of Samruddhi S. Thawari, says that steel is a popular construction element used in the entire building field. The main function of the structure is to shape a foundation, essentially the portion of the structure that keeps everything together. Steel is one of the most friendly and 100 percent recyclable natural products. It has evolved in structural architecture, primarily due to the need of earthquakes. The required design specifications, particularly for highly loaded structures, can not be met with the use of the available ISMB steel parts, as inertia and cross sectional times play an important role.

CHAPTER 3

DESCRIPTION OF ROOF TRUSS

3.1 ROOF TRUSS

Trusses constructed of steel are widely used in industrial building. We are pre-manufactured to order and are designed in an open platform style. They are primarily axially loaded members that withstand external loads more effectively because the cross-section is almost equally stressed. They are commonly used, especially in order to fill significant gaps. Thrusts are used to withstand gravity loads in single-story commercial structures, long floors and roofs. The downside of utilizing steel trusses for design is that they are heavier than timber so more open space is available within a house. Ideal for barns, large storage areas and industrial buildings.

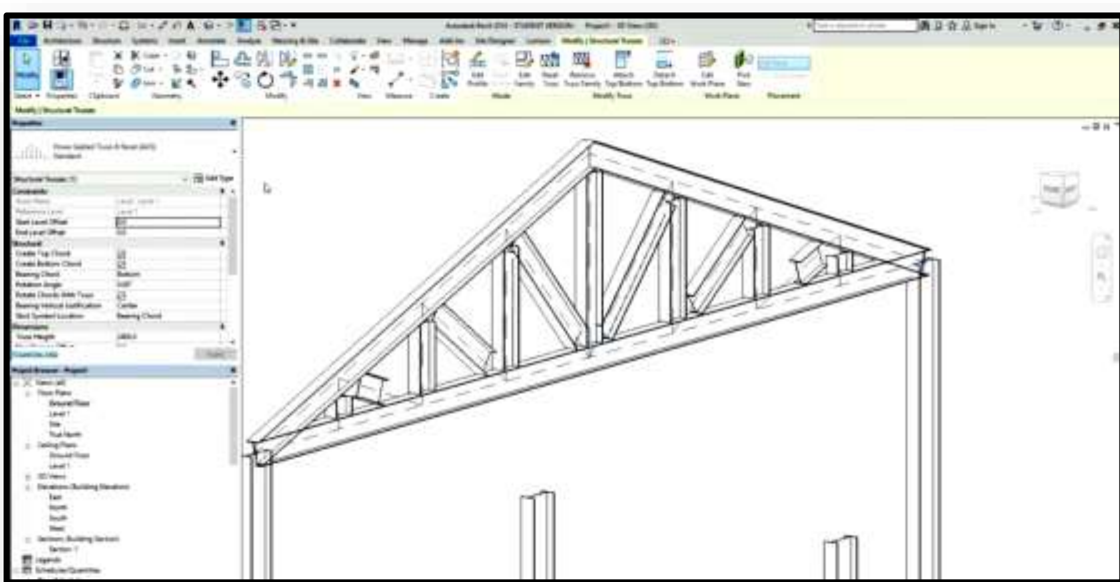


Fig.3. A 3D truss model showing internal components like arrangement of truss and connection between bays

3.2 ANALYSIS OF ROOF TRUSS

It is usually thought that truss members are linked together in order to move only the axial forces and not moments and shears from one component to the neighboring members (they are called locked joints). This is presumed that the loads just work at the trusses nodes. The trusses can be supplied over a continuous line, assisted merely between the two end supports, in which case they are typically calculated dynamically. These trusses may be manually evaluated by joining system or segment process.

From the study focused on a pinned common theory, one obtains only the axial forces in the different trusses leaders. However, in actual configuration the trusses members are connected by more than one bolt or by welding, either directly or by wider end gussets. Additionally, some of the members, especially chord members, may be continuous over many nodes. Generally, these joints promote not only translation consistency but also rotation flexibility with representatives meeting at the joint. As a consequence, in addition to axial force the trusses leaders undergo bending moment. Additionally, the loads can be placed between the trusses nodes, allowing members to turn. Such stresses are dubbed secondary stresses. The secondary bending stresses can also be induced by the members' unusual relation at the joints. An indeterminate structural model may be used to evaluate trusses for the secondary moments and thus the secondary strains, typically utilizing machine software that is discussed in this article.

Owing to joint rigidity, the severity of the secondary stresses depends on the rigidity of the joint and on the toughness of the members meeting at the joint. If the slender chord member's ratio exceeds 50 and the slim ratio of web members exceeds 100, then secondary stress in roof trusses can usually be ignored. The secondary stresses cannot be ignored when caused by the imposition of loads on members within nodes and when members are eccentrically bound together.

3.3 DIFFERENT TYPES OF L0ADS

3.3.1 DEAD L0AD (D.L)

The dead load on the roof trusses of single-story commercial buildings usually consists of the Dead load of cladding and the dead load of purlins, the self-weight of the trusses of addition to the weight of brackets etc. Unique dead loads including dead loads of trusses may even contribute to fading lots of roof trusses, different conduits and fan weight, etc. If the transparent span duration (column free span duration) decreases, the self-weight of the gable frames that withstand the moment rises significantly. In these situations roof trusses are more cost-effective. By introducing composite slabs with profiled steel sheets, the dead loads of floor slabs can be minimized considerably.

3.3.2 LIVE L0AD (L.L)

Live loads on the roofs are the gravity of building, repairs and dust load etc. and the amount is taken under IS: 875-1975. Additional special live loads, such as snow loads in extremely cold environments, may need to be addressed for crane live loads in trusses supporting monorails

3.3.3 WIND L0AD (W.L)

If the roof slope is too high, the pressure on the roof trusses will typically raise the force perpendicular to the roof because of the wind flowing through the roof. Therefore the wind load on the roof truss typically works counter to the load of gravity, and its amplitude can be greater than the load of gravity, forcing the forces in the truss members to change.

3.3.4 EARTHQUAKE L0AD (E.L)

As the strain of a building with the earthquake depends on the density, earthquake loads do not generally control the architecture of light-duty industrial steel constructions. Typically wind loads govern. However, the load of an earthquake can govern the design in the case of industrial buildings which have a large mass on their roofs or uppers. The loads are determined according to IS: 1893-2002.

3.4 VARIOUS ELEMENTS OF INDUSTRIAL STEEL BUILDING

Various elements are in the chosen model structure,

- Truss members
- Purlins
- Beams connecting the trusses to form bays.
- G.I corrugated color profiled sheets.
- Reinforced concrete column.
- Primary steel frame

- Roof bracing
- Side Rails
- External walls of 30cm thick brick.

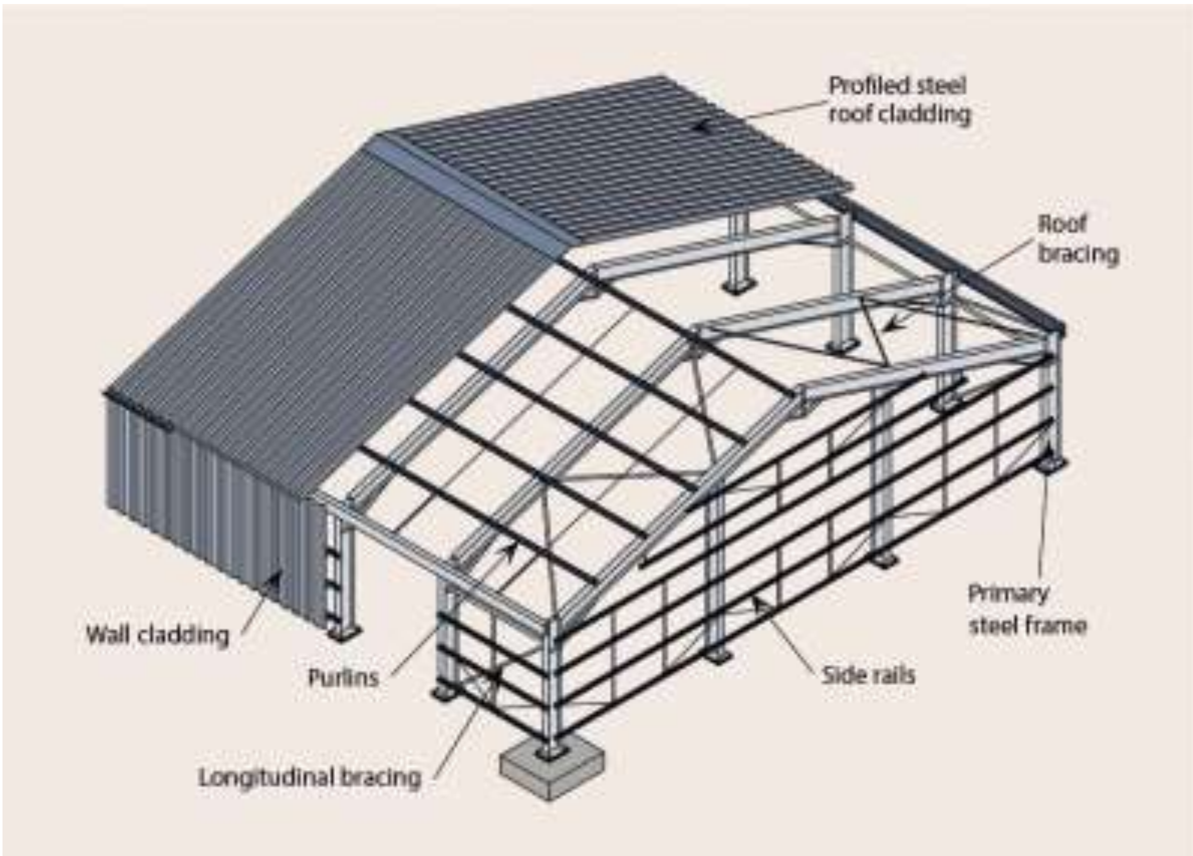


Fig.4. Competent of Industrial Building.

3.5 COMPUTER MODEL

Based on customer criteria, a model was picked. Autodesk REVIT is used to develop a three-dimensional structural space truss model for the building. Truss Geometry is designed and prepared, which is followed by analysis and design. The Autodesk REVIT image files are shown in Fig. 5, 6 & 7.

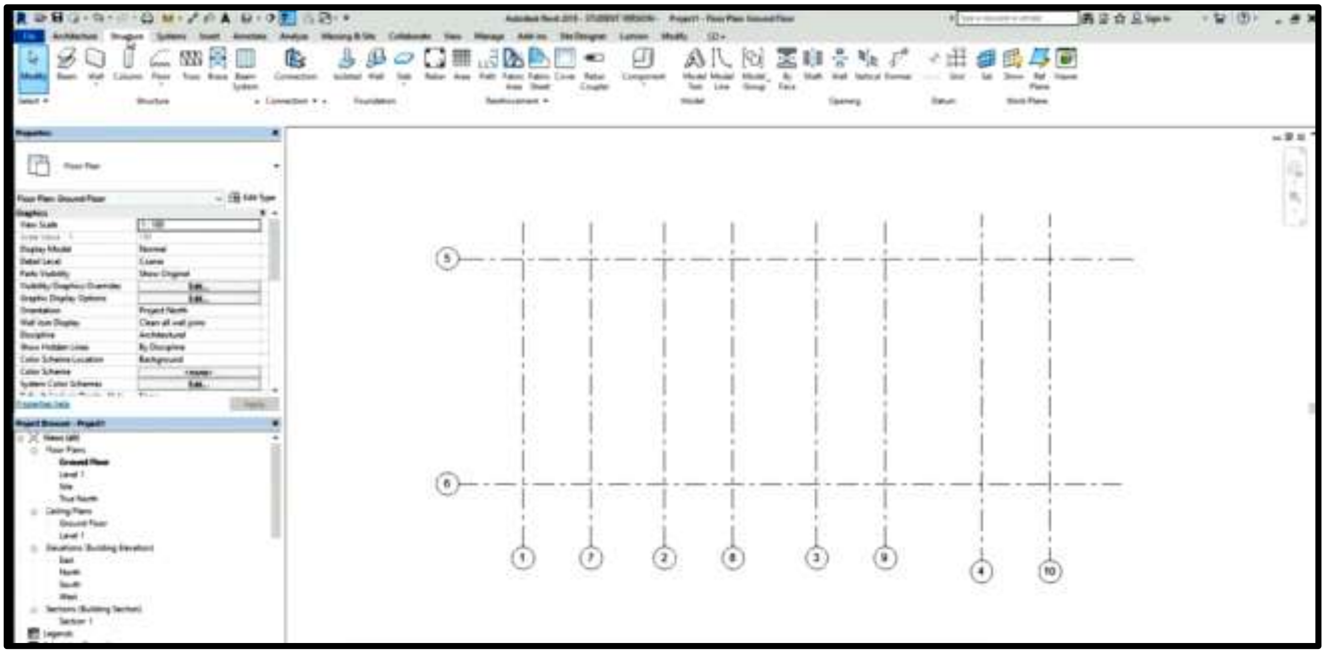


Fig. 5 Grid Plan Of Structure In Revit

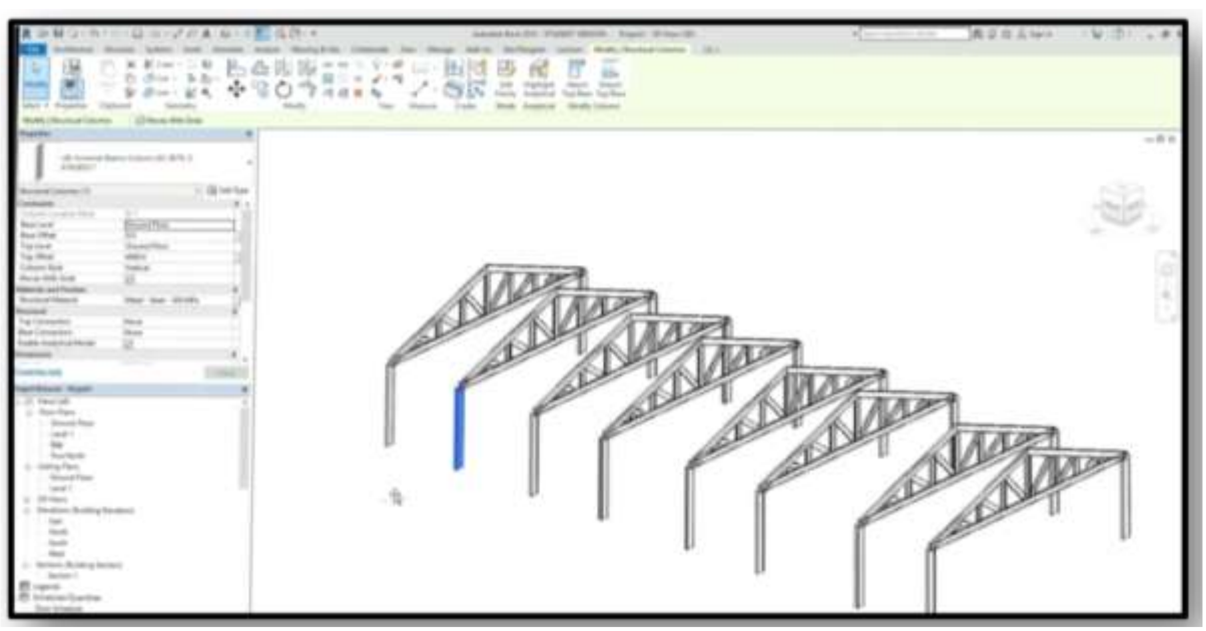


Fig. 6. 3D Elemental View of Entire Industrial Structure

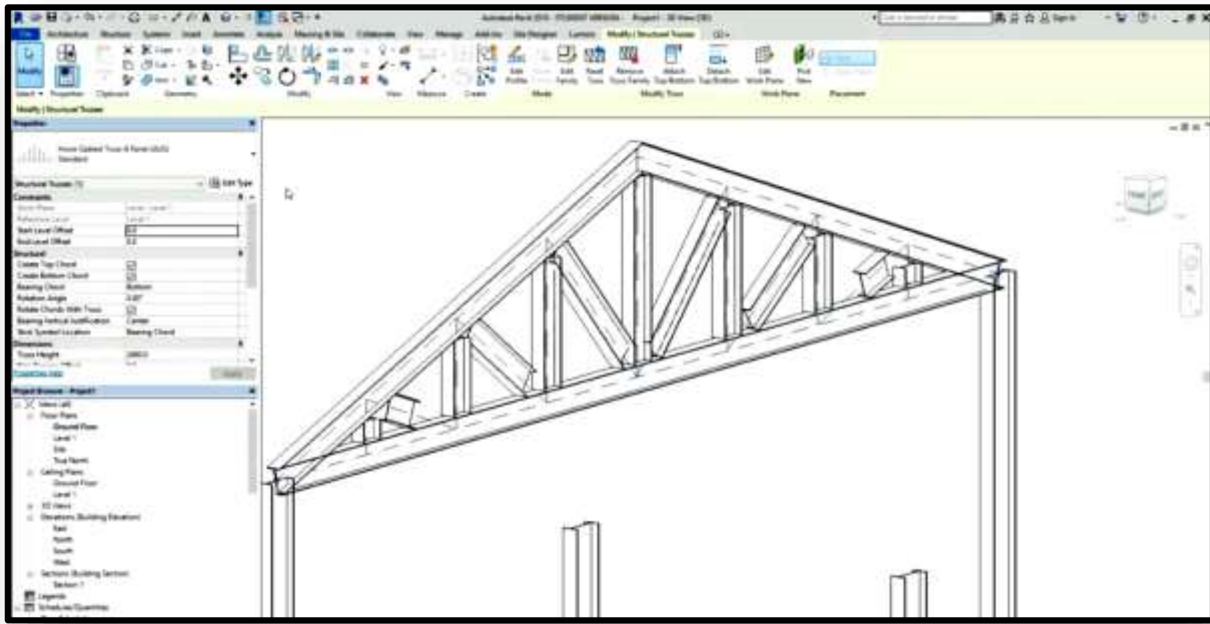


Fig 7. Truss Element in Autodesk Revit

3.6 ASSUMPTIONS AND ANALYSIS CRITERIA

The research process and the study parameters are in line with the concept basis.

The overall construction in a steel code of practice 800-2007 (third revision) and the columns and footing of the roofing frame are based on IS: 456-2000 plain, reinforced concrete practice code (fourth revision).

3.7 ANALYSIS SOFTWARE

The structural research is carried out using the Autodesk Revit BIM program. The usage of the Revit Building Information Model provides structural companies an interactive modelling framework for research and documents – such that the construction planning and data are organized, reliable and detailed.

These technical measurements shall be described in the system of international (SI) units as seen below:

- Length - Meters (m)
- Force - kilo Newton (kN)

- Moment- kilo Newton meter (kN-m)
- Stress - MPa or N/mm2

The structural steel members material properties are described 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 x 10^6/^{0}C

All structural components used are available in Indian market by TATA structure according to the structural steel requirements.

Design stress shall not be greater than the permissible stresses defined by IS 800 according to the corresponding grade and profile of the element in steel and dimensions.

3.8 ENVIRONMENTAL CONDITIONS

3.8.1 WIND LOAD

In order to evaluate the following wind speeds, average wind speed of one hour shall be used.

DIRECTION	WIND (m/sec)
FOR ALL DIRECTIONS	47

Table 1.Wind speed for operating conditions

3.8.2 LEVEL OF WATER TABLE

The water table is believed to be just below the floor and does not impact the structure.

3.8.3 TYPES OF FOUNDATION SOIL

The surface is considered to be hard ground.

3.8.4 LOCATION AND SURROUNDING

LOCATION	SURAJPUR ,GREATER NOIDA
SURROUNDINGS	PLAIN AREA WITH BUSHES LESS THAN 0.5m HT.

Table 2.Location and Surroundings

3.9 LOADS AND DIFFERENT COMBINATIONS

3.9.1 L0ADS

Structural loads or actions are movements, deformations or accelerations that are transferred to a system or to its occupants. Loads induce structural stresses, deformations, and displacements. Assessment of their results is performed using quantitative empirical techniques. Excess loading or overloading may cause structural collapse, so that possibility should be recognized either in the design or strictly regulated. Structural loads are also determined by engineers based on written legislation, contracts, or requirements. Accepted quality criteria are used for the preparation and examination of acceptances.

Building codes allow buildings to be built and installed to safely withstand any acts they are likely to encounter during their service life, while being fit for use. Maximum loads or behavior for styles of buildings, regional areas, use, and construction materials are defined in these building codes.

By their original source, structural loads are classified into groups. Of example, there is no distinction in dead or live loading in terms of the real load on a system, but the separation occurs as follows for use in safety measurements or for ease of study on complex models.

Building codes recommend that loads be raised by load factors where construction strength is greater than normal loads is needed for structural architecture. These factors roughly reflect a relation between theoretical strength of the system and the actual duty load. It leads to the desirable degree of systemic reliability based on probabilistic studies taking into account the initiating cause of load, recurrence, delivery and the static or fluid existence.

3.9.2 TYPES OF L0ADS

3.9.2.1 DEAD L0ADS

Dead load involves time-constant loads including structural weight and fixed fittings including walls, plasterboard or furniture. Often known as static loads are dead loads.

The builder often understands the scale of dead loads fairly well as they correspond directly to building material density and quantity. These have a low variance, and the designer himself is normally responsible for the specifications of these components.

Dead loads are permanent loads that do not change in the structure's life. They are,

- Building's Self-weight.
- Built-in material: walls, floors, roofs, ceilings and permanent frameworks.
- Standard equipment: furniture, fittings, electrical cables, pipes for ventilation, vent air network.
- Partitions, fixed and movable parts of building.
- Material stored inside building

If the configuration shifts substantially, dead loads should be reassessed and a fresh structural review should be carried out.

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).

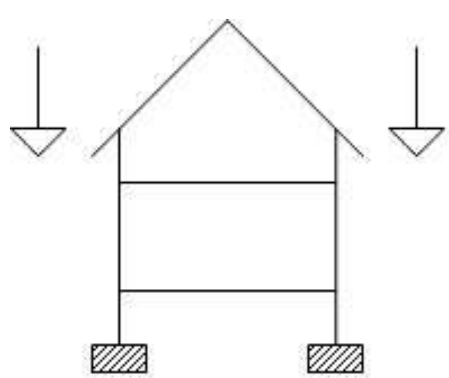


Fig.8. Direction of Dead load

3.9.2.1.1 MOVABLE TEMPORARY PARTITION

The real weights measured, but not less than, of the UDL of 0,5 kPa for the region being regarded, for the self-weight of moving partitions. Attention is extended to the elimination of permanent objects that are not integral sections of the system, such as water tanks, storage products, service facilities, partitions and related items.

3.9.2.1.2 LOAD FACTORS FOR DEAD LOADS

Because the variation of dead loads is negligible of structural life, no variation in dead loads can be taken into account. Dead loads are only taken into account when designing members to different limit states to generate factored load combinations.

3.9.2.2 LIVE L0ADS

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 e.g., a factory is far more likely than a residential bedroom to have greater loads. Whitewashers in a stadium can see greater loads than on a pitched construction roof.

Generally defined live loads are either represented as uniformly distributed area loads, or as point loads spread over specific areas.

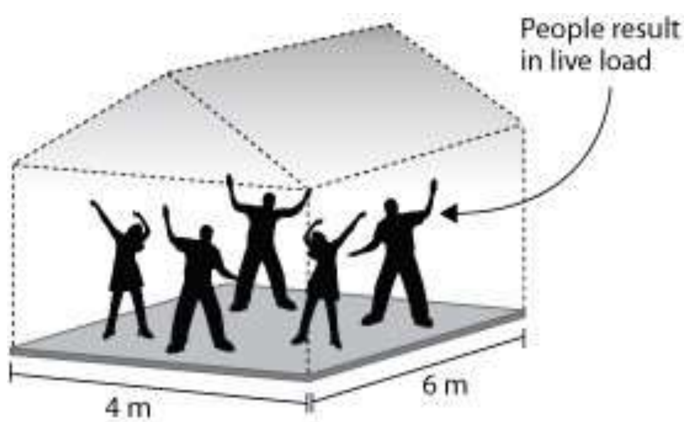


Fig.9. Live load in a Building.

3.9.2.2.1 UNIFORMLY DISTRIBUTED LOADS

The loads are equally dispersed for parts of the system and can be spread relatively consistently around broad areas (size of one or more of the rooms). Where the live load Q moves from one space to another, the review shall be done to take into account the most adverse load cases :-

Live load factor for all the spans.

- live load factors for two adjacent spans.
- live load factor for 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/N^2 for each meter of storage height.

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

3.9.2.2.2 CONCENTRATED LOADS

There is the capacity for high condensed load (like a huge copying machine) in certain occupancies, such as office spaces. This room can also be built for uniformly distributing loads but it is impossible that both the uniformly dispersed load and the broad clustered load would concurrently fill the area.

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) In a location which gives the most adverse impact at its known position or when its position is unknown.

(b) Spread in the particular domain region or unknown to the specific field.

3.9.2.2.3 ROOF AND SUPPORTING ELEMENTS

Roofs, rather than routine maintenance and minimal fixes, are deemed non-accessible. When roofs are mostly available and used for flooring operations, floors are handled. Roof load calculated according to Table 2 of IS 875 (part2) for imposed loads as,

IL on roof = 0. 75 — 0. 52y^2

Where, y = 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

3.9.2.3 WIND LOADS

The pressure of the wind on the structure depends on the location of the structure, the height of the structure above the ground level and also on the shape of the structure. The code gives the systems in various sections of the world specific wind power. The wind pressure is also significant .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 up to 10 m in height, the strength of the wind pressure as defined in the code can be decreased by 25% for the measurement of the stability and for the construction of the structure as well as the cladding. This reduction can only be rendered for stability measurements and column design in buildings above 10 m and up to 30 m in height.

The overall weight on the walls or rooftops of an industrial building relies both on the outer wind pressure and on the inner wind pressure. The wind pressure within the buildings depends on their permeability. The internal air pressure may be ignored for buildings with a low degree of permeability. The interior pressure can be ± 0.2p for buildings with standard permits. Here '+' means pressure and '-'suction, 'p' means the fundamental wind pressure. If an openings surpass 20% of the wind pressure in the house. 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 unit wind pressure 'p' at the wall is taken as 0,5p wind pressure and 0,5p lean surface suction. The revolving wall undergoes a strain of 0.5p and the leaking wall under the suction of 0.5p because the walls are an enclosure. The overall weight on the walls also depends on the weight of the internal soil.

Built wall pressure = 0.5p for buildings with low permeability

Built pressure on wall = 0.7p for buildings with natural permeability.

Built pressure on wall = p for buildings with wide openings.

Wind loads on roofs

Through multiplying fundamental pressure p through factors defined in Table 7.1, the natural pressure to the slope of the roof is achieved. The table demonstrates also the influence of the inner pressure generated by the permeability of the walls or roof opening. The maximum exterior roof wind pressure may be calculated as -0.6p if the wind moves in line with roof tops 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 the surface, the wind forces act in the direction of the wind on the surface. The 'Wind Drag' is this power. When the wind flows naturally to the ridges, the wind drag is 0.05p in the case of urban structures 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.

Wind pressure on roofs (Wind normal to eaves) Sums of external and internal pressure

Roof of pitch	Zero Permeability External Pressure		Normal Permeability $p_1^* = +0.2p$		Normal Permeability $p_1^* = -0.2p$		Large openings $p_1^* = +0.5p$		Large openings $p_1^* = -0.5p$	
	Windward	Leeward	Windward	Leeward	Windward	Leeward	Windward	Leeward	Windward	Leeward
1	2	3	4	5	6	7	8	9	10	11
0	-1.00	-0.50	-1.2	-0.70	-0.8	-0.30	-1.5	-1.00	-0.5	0.00
10	-0.70	-0.50	-0.9	-0.70	-0.5	-0.30	-1.2	-1.00	-0.2	0.00
20	-0.40	-0.50	-0.6	-0.70	-0.2	-0.30	-0.9	-1.00	+0.1	0.00
30	-0.10	-0.50	-0.3	-0.70	+0.1	-0.30	-0.6	-1.00	+0.4	0.00
40	+0.10	-0.50	-0.1	-0.70	+0.3	-0.30	-0.4	-1.00	+0.6	0.00
50	+0.30	-0.50	+0.1	-0.70	+0.5	-0.30	-0.2	-1.00	+0.8	0.00
60	+0.40	-0.50	+0.2	-0.70	+0.6	-0.30	-0.1	-1.00	+0.9	0.00
70	+0.50	-0.50	+0.3	-0.70	+0.7	-0.30	0	-1.00	+1.00	0.00
80	+0.50	-0.50	+0.3	-0.70	+0.7	-0.30	0	-1.00	+1.00	0.00
90	+0.50	-0.50	+0.3	-0.70	+0.7	-0.30	0	-1.00	+1.00	0.00

p_1^* => internal pressure

Table. 3. Wind loads on roofs

The wind loads are calculated using **IS: 875(part3)** as

Wind Pressure = 0.6 X V2

Where Vz= Design wind speed

Vz = k1k2k3Vb

k1 = probability factor

k2 =Terrain and height factor

k3=Topography factor

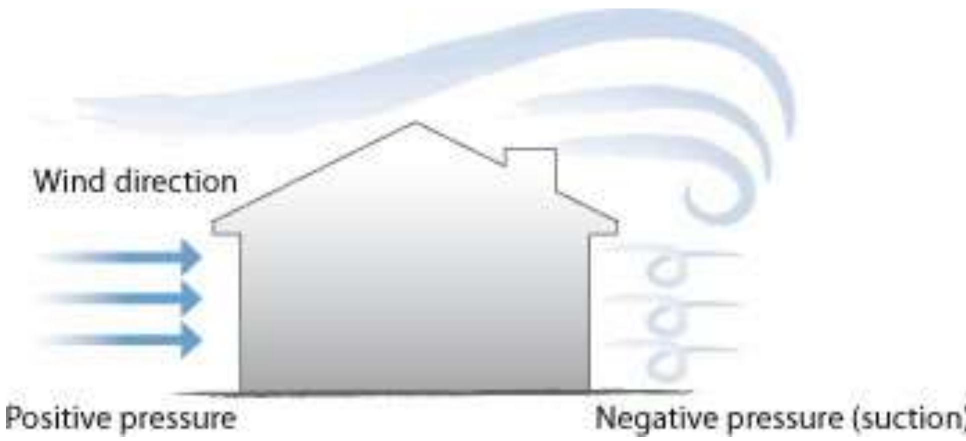


Fig.10 Effect of wind on building

3.9.2.4. SEISMIC LOADS

Single-story commercial buildings are typically controlled by loads of wind rather than earthquake. The roofs and walls are light and sometimes sloping or raised as well as the structures are wind allowed, which allows the roof to be elevated. However, testing for wind and earthquakes in a building is still secure.

Earthquake loads differ in various respects from wind loads and so earthquake designs are also very different from wind loads and other gravity loads design. Normal earthquakes are very strong and thus elastic behavior under mild earthquakes is typically assured and ductile in major earthquakes. Their consequences are very significant. Steel is naturally ductile and so only mild earthquakes are used to measure stresses. The IS 1893 code will do that.

In compliance with that code a horizontal seismic coefficient time should also be added as an equal load for a static earthquake and the system in conjunction with other loads defined in IS 800 should be tested for protection under this load. The following are the combinations:

1. 1.5 x (DL + IL)
2. 0.9 x DL + 1.5 x EL
3. 1.5 x (DL + EL)

4. 1.2 x (DL + IL + EL)

The horizontal seismic coefficient Ah takes into account the position of the system by zone factor Z, the strength of the system by factor I and the ductility by factor R. It also calls the stability of the framework base method by means of the Sa / g acceleration ratio, which is a function of the normal time span T. This last ratio is given as a map called the response spectrum. The horizontal seismic coefficient Ah is given by

$$An = \frac{ZISa}{2Rg}$$

Where, Z = Zone factor corresponding to the seismic zone obtained from a map

I = Importance factor,

R = Response reduction factor,

S_a = Spectral Acceleration Coefficient

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

Table 4. Zone Factor Z

For industries using hazardous materials and fragile products the importance factor may be taken as 1.5 Yet it can be called 1.0 for most enterprises. In buildings where specific information pursuant to section 12 of IS 800 were not known, the Response reduction factor R may be taken as 4.

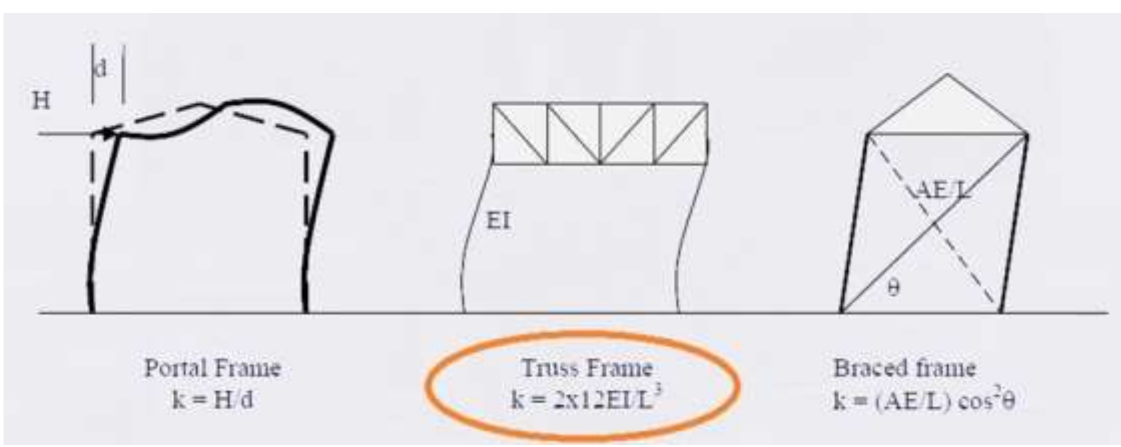


Fig. 11 Lateral Stiffness of various building configurations

The normal time duration T is rather critical and should be accurately measured. For one floor frameworks, it can be estimated to be $T = 2\pi\sqrt{(k/m)}$, where k is the lateral (horizontal) rigidity, and m is generally considered to reflect the roof weight, plus 50% of live load separated by the total roof weight, the acceleration due to gravity g. Guidelines for calculating k in some simple cases are given in Fig. 7.5.

Finally, the acceleration ratio Sa/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.

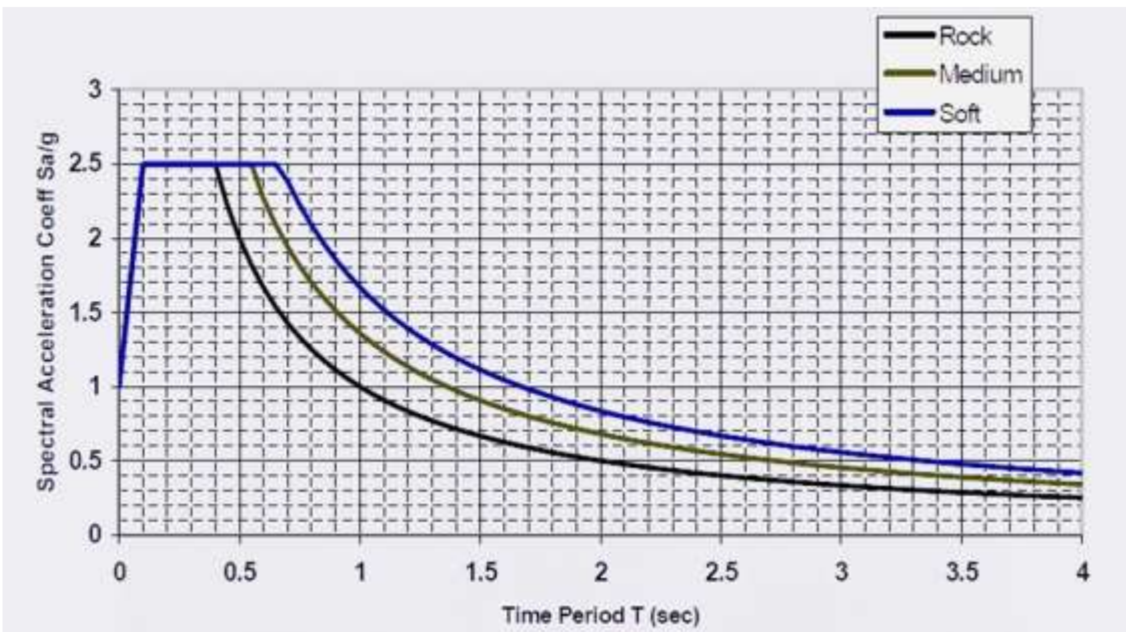


Fig 12. Response Spectrum for 5% damping

7.1.2. LOAD CASE DETAILS

Load Case No	Load Case name	Load value
1	EQX	
2	EQ-X	
3	EQZ	
4	EQ-Z	
5	DL	0.131kN/m
6	LL	0.741kN/m
7	WLX	1.471 kN/m^2
8	WL-X	1.471 kN/m^2
9	WLZ	1.471 kN/m^2
10	WL-Z	1.471 kN/m^2

Table 4. Load case details

7.2. LOAD COMBINATIONS

The configuration load variations are the various varieties of potential load cases to be built for the system. In case of IS 456-2000, the following load combinations may be needed if a system is subject to death (D), living (L), live patterns (PL), snow (S), Wind (W), and earthquake (E). (IS 36.4, Table 18):

- 1.5xDL (IS 36.4.1)
- 1.5xDL + 1.5xLL
- 1.5xDL + 1.5xSL (IS 36.4.1)
- 1.5xDL + 1.5x(0.75 PL) (IS 31.5.2.3)
- 1.5xDL + 1.5xWL
- 0.9xDL + 1.5xWL
- 1.2xDL + 1.2xLL + 1.2xWL
- 1.5xDL + 1.5xLL + 1.0xWL (IS 36.4.1)
- 1.5xDL + 1.5xEL
- 0.9xDL + 1.5xEL

- 1.2xDL + 1.2xLL+ 1.2xEL
- 1.5xDL + 1.5xLL+ 1.0xEL (IS 36.4.1)
- 1.5xDL+ 1.5xLL+ 1.5xSL
- 1.2xDL+ 1.2xSL + 1.2xWL
- 1.2xDL+ 1.2xLL + 1.2xSL + 1.2xWL
- 1.2xDL+ 1.2xSL +1.2xEL
- 1.2xDL+ 1.2xLL + 1.2xSL + 1.2Xel

Of the above load variations, the architecture of this building just takes into account the potential varieties. The following table describes them:-

Table 5.Various combinations of loads considered

LOAD COMBINATION NUMBER	LOAD COMBINATION NAME
7	1.5(DL+LL)
8	1.2(DL+LL+EQX)
9	1.2(DL+LL+EQ-X)
10	1.2(DL+LL+EQZ)
11	1.2(DL+LL+EQ-Z)
12	0.9DL+1.5EQX
13	0.9DL+1.5EQ-X
14	0.9DL+1.5EQZ
15	0.9DL+1.5EQ-Z
16	1.5(DL+WLX)
19	1.5(DL+WLZ)

CHAPTER 4
ANALYSIS AND DESIGN

4.1 ANALYSIS

The results of the REVIT BIM analysis will be provided 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.

Analysis and Result are under process.

4.2 DESIGN

4.2.1 DESIGN OF PLINTH FLOOR SLAB

4.2.1.1 DESIGN OF ONE-WAY SLAB:-

The design procedure for a single-way slab is the same as for beams. However, the amounts of reinforcing bars are for one meter 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: Preliminary slab depth selection:-

The depth of the slab shall be assumed from the span to effective depth ratios.

Step 2: Design loads (DL), bending moments (B.M) and shear forces (S.F):-

Total factored (design) loads are to be calculated by including the approximate dead load of the concrete, the load of the floor cover, specified or expected live loads, etc. after combining each of them with the respective partial safety variables. Subsequently, the specification of positive and negative bending moments and shear forces shall be calculated using the respective coefficients set out in Tables 12 and 13 of IS 456.

Step 3: Check for Slab's effective and total depth:-

The successful depth of the slab shall be calculated through the usage thereof

$$2Mu,_{lim} \quad = R_{lim} \; x \; b \; x \; d^2$$

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

The overall depth of the slab is then calculated by applying the correct nominal coverage (Table 16 and 16a of Clause 24.6 of Clause 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 determination of the depth of the slab must be rendered only after the shear has been tested.

Step 4: Using Depth of slab for determination of shear force (S.F):-

In principle, if the shear strength of the concrete is known, the extent of the shear force may be verified. Because this relies on the percentage of tensile strengthening, the product shear strength is considered to be the lowest percentage of steel. The value of cτ 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

Steel reinforcing region in the form of a one-way slab utilizing Eq can be calculated:-

$$M_u = 0.87 \, fy \, Ast \, d \, \{1\text{-}(Ast)(fy)/(fck)(bd)\}$$

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 steel area so defined will be tested if it is at least the minimum steel area alluded to in paragraph 26.5.2.1 of IS 456.

Alternatively, SP-16 tables and maps can be used to calculate the depth of the slab and the associated steel field. Tables 5 to 44 of SP-16, representing a broad variety of grades of concrete, and Map 90, shall be used to assess the strength and strengthening of the slabs. Tables SP-16 take into consideration the average bar diameter not approaching one-eighth of the slab width. Zeros in the top-right corner show the region in which the reinforcement percentage would exceed slim . Similarly, at the bottom left corner of the reinforcement area zeros indicate the area that is less than the minimum specified in the code. Therefore, for an approved maximum diameter of bars or the calculated steel area over a minimum steel area while using tablets and diagrams of SP-16, no separate inspection is required. The amount of steel strengthening over the broad span shall be the minimum steel quantity in compliance with IS 456, c.26.5.2.1.

Step 6: Selecting Bar Spacing C/c and its diameter (clas.26.5.2.2 and 26.3.3 of IS 456)

The bar diameter and spacing should be calculated in compliance with IS 456 clauses 26.5.2.2 and 26.3.3. As mentioned in step 5, the tables and CHARTS OF SP-16 can be used to avoid this step.

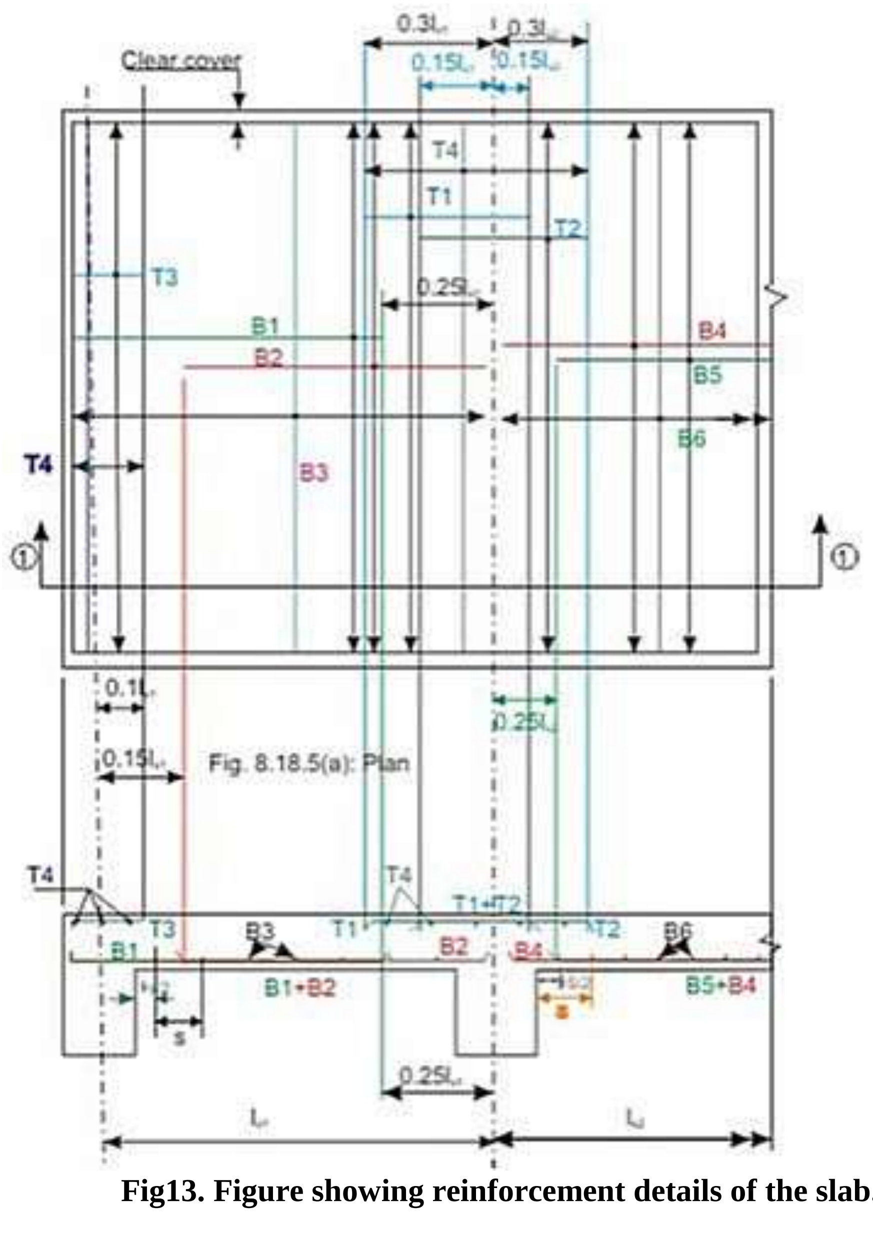


Fig13. Figure showing reinforcement details of the slab.

4.2.2 DESIGN OF FOOTING

4.2.2.1 BASIS OF DESIGN IN FOOTING

Footing is classified as one-way footings below walls and two-way footings below columns. The first step in design is to calculate the necessary area from the formula.

$$\textbf{Area of footings} = \frac{\text{service load on column or wall above}}{\text{sarfe bearing capacity of soil}}$$

Having thus determined the size of the footings, its structural design is carried out by using factored loads and principles of limit state design s already discussed in the case of other R.C members. The thickness of the foundation and its strengthening are the main elements to be built. 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 75to 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.

4.2.2.2 DESIGN PRINCIPLES

The footings should not be exceeded for the maintenance of applications and loads, moments and forces and for stable bearing ability. The thickness at the edge on the ground of R.C.C. shall not be lower than 150 mm for basements on soil. The higher BM used in the design of an isolated concrete base which supports a column, pedestal or walled structure shall be computed. The diagonal critical component shall be taken from a hard ground and shall not exceed the nominal shear stress, at a distance equal to the effective depth of the face of the column.

4.2.2.3 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	=1.417 KN m
Factored moment 1.5 X 1.714	=2.126 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/m^2

Size of the footing:

Area of the footing = 600/300 =2m^2

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 = 1.625O

Hence, 1.625B = 2m^2

Hence,B = 1.23 m say 1.25Om

L = 2 m

Provide size of footing = 2 m x 1.25O m

Stress on the footing $= \frac{P}{B} \pm \frac{My}{I}$ <3OOkN/m^2

I = 1/12x2x1.25^3

= O.33 m^4

Y = 1.5/2 = O.75m

stress on the footing $= \frac{550}{1.25} \pm \frac{2.127\times\cancel{0.75}}{0.33}$

= 275 ± 4.835

P_{max}=275 + 4.835 = 279.835kN/m^2

< 3OO kN/m^2

P_{min}= 275 - 4.835 = 271.O45 kN/m^2

Hence design footing can bare a maximum pressure of 279.834kN/m^2

Projections beyond column faces = (2–0.750) / 2 **= 0.625**

Check for two way shear

As per IS 456-2000, the critical section for two way shear at a distance of $^{d}/_{2}$ from the face of the column.

Taking a section at d/2 around column, we get

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

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

= 442.2 KN.

$$\text{Nominal shear stress } = v_v = \frac{V_u}{B \times d}$$

=1391.4*1000(2*(300+540+750+540)*540)

=0.59 N/mm^2

Shear strength of M$_{20}$ concrete

$\tau^u_c = k_s\tau_c$

$k_c = (0.5 + þ_c)$

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

=38O/75O = 0.5O7

K_s = 0.5 + 0.507

= 1.007 < 1.000

K_s =1.0

$\tau^u_c = \tau_c = 0.25(f_{ck})^{0.5}$

= 1.14 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.

$$V_{max} = \frac{p*B(L–a–2*d)}{2}$$

$= 284*1.8*(3.6-0.75-2*.54)/2$

$=503.53$ kN

Nominal shear stress $v_v = \frac{V_u}{bd}$

$$= \frac{503.53*1000}{1800*540}$$

$= .46$ N/mm^2

Allowable shear stress, can be calculated by percentage of steel.

$$P = \frac{100*3455.8}{1800*540} = 0.4\%$$

For P = 0.4%

$\tau_c = 0.47$ N/mm^2

Therefore $v_v < \tau_c$

Hence safe.

4.2.3 DESIGN OF STEEL TRUSS

4.2.3.1 GEOMETRY CALCULATION:-

Consider a pitch, $\frac{1}{6}$ of span.

Height of truss = $\frac{1}{6}$ x 25 = 4.166 m.

Spacing of truss = $\frac{L}{5} = \frac{25}{5}$ = 5 m.

Slope of top Chord = $\tan^{-1}\frac{4.166}{12.5}$ = 18.43°

Pitched Length of top chord=$\sqrt{12.5^2 + 4.166^2}$ = 13.175m

Distance between Purlins =$\frac{13.175}{4}$ = 3.29m.

Pitched Area = (pitched lenght x truss spacing) x 2

= (13.175 x 5) x 2 = 131.75 m^2

Plan Area = 25 x 5 = 125 m^2

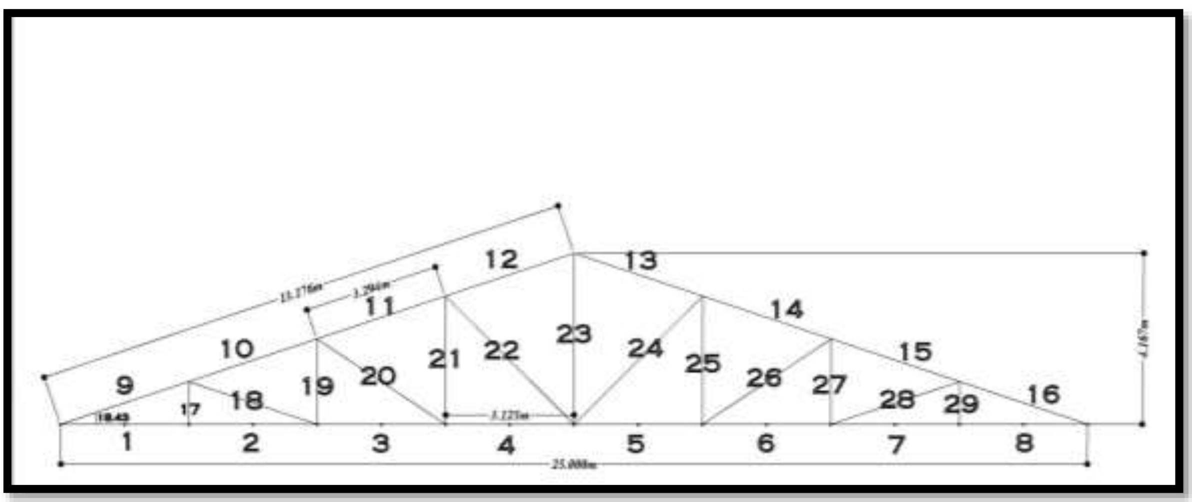


fig 14 Group member and Geometry of Steel Truss.

4.2.3.2 DIFFERENT LOADING CALCULATION:-

Dead Load (DL) Calculation:-

Self-weight of CGI sheet = 150 N/m^2

Self-weight of Purlin = 100 N/m^2

Total self-weight of Purlin = 10x5x100 = 5000 N

Total self-weight of truss on Plan Area = 133.33 x 25 x 5 = 16666.25N

Total self wight of truss CGI sheet & Wind Bracing on Pitched Area = $(150 + 20) \times 131.75$ = 22397.5 N

Total D.L = a. + b. +c. = $5000 + 16666.25 + 22397.5 = 44063.75$ N

Dead loads at every Panel Node = $\frac{44063.75}{8}$ = 5508 N.

Dead loads at Ends Nodes = $\frac{5508}{2}$ = 2754 N.

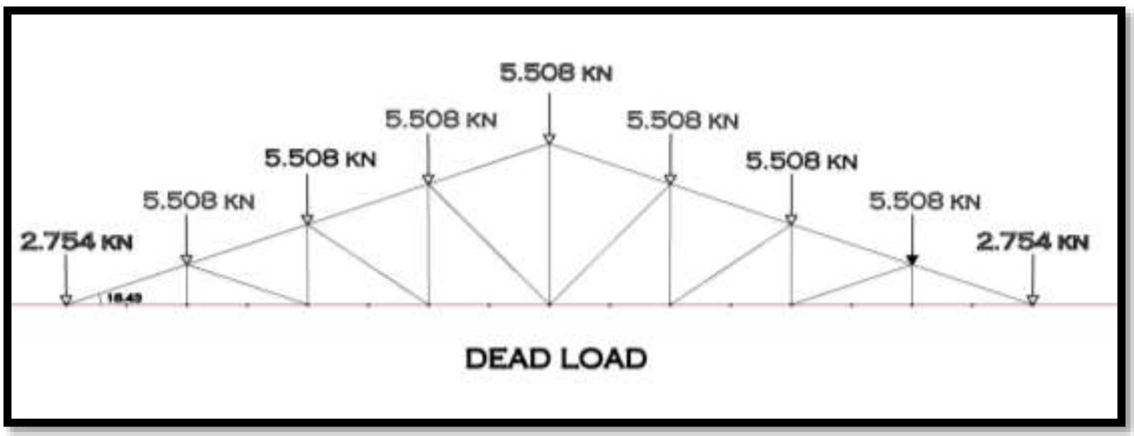


Fig 15. Dead Load on Each Panel Points.

<u>Live Load (DL) Calculation</u>:-

Imposed load on truss = 750- 20x (18.43-10) = 581.4 N/m^2

Live load on the truss = .66 x imposed load x plane area = .66 x 581.4 x 125 = 48450 N.

Live load on each panel points = $\frac{48450}{8}$ = 6056.25 N.

Live load on end panel points = $\frac{6056.25}{2}$ = 3028 N.

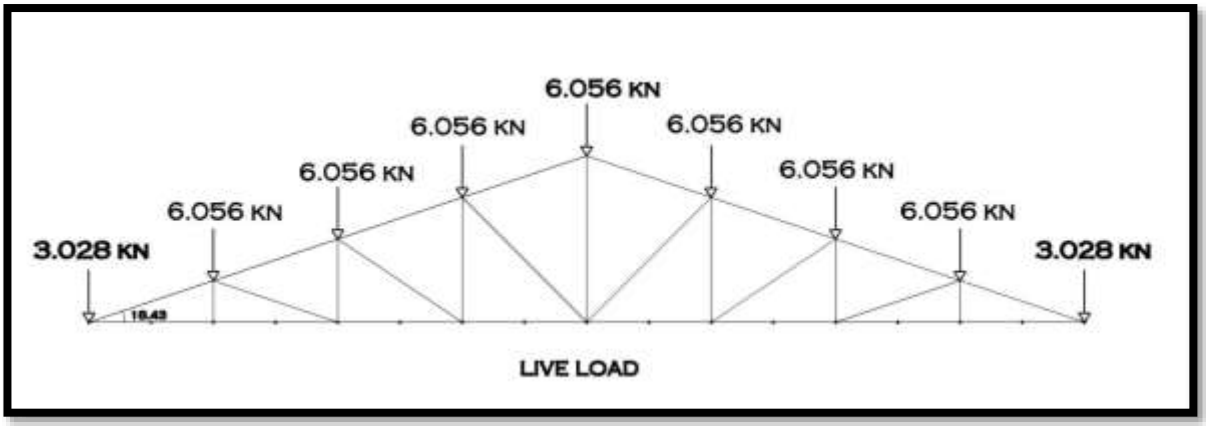


Fig 16. Live Load on Each Panel Points.

Wind Load (WL) Calculation:-

Design wind Speed (Vz) = Vb x K1 xK2 x K3 x K4.

Where Vb = basic wind speed = 47 m/s.

K1 = Probility factor

K2 = terrain/ height/ structure factor

K3 = Topograghy Factor , it is taken as unity.

K4 = Importance factor for cyclonic region.

Vz = 47 x 1 x 1 x 1 x 1.

Vz = 47 m/sec.

Basic Wind Pressure = 0.6 $(Vz)^2$ = 0.6 x 47^2 = 1325.4 N/m^2

Wind Load (F) on Building = $F=P_z \times (C_{pe} \pm C_{pi})$

<u>Assuming Wind Normal to Ridge</u>

Calculation are given below:

Net pressure calculation:-

For windward side slope:-

-526 + 0.2x1325 = -261

-400 – 0.2x 1325 = -135

	Windward side	Leeward side
10	-1.2	-0.4
18.43	-0.526	-0.4
20	-0.4	-0.4

For leeward side slope:-

-526 - 0.2x1325 = -791

-0.4 + 0.2x 1325 = -135

<u>Assuming Wind parallel to Ridge</u>

Calculation are given below:-

Net pressure calculation:-

For leeward side slope:-

-0.6 + 0.2x1325 = -531

-0.715 – 0.2x 1325 = -1210

	Windward side	Leeward side
10	-0.8	-0.6
18.43	-0.715	-0.6
20	-0.4	-0.6

For windward side slope:-

-0.6 - 0.2x1.325 = -1060

-0.715 + 0.2x 1.325 = -135

Total wind force = Sloping Area x Intensity of load = 131.75 m^2 x 1210 = 159417.5 N

Wind load per panel points = $\frac{159417.5}{8}$ = 19927 N.

Wind load per end points = $\frac{19927}{2}$ = 9963.6 N.

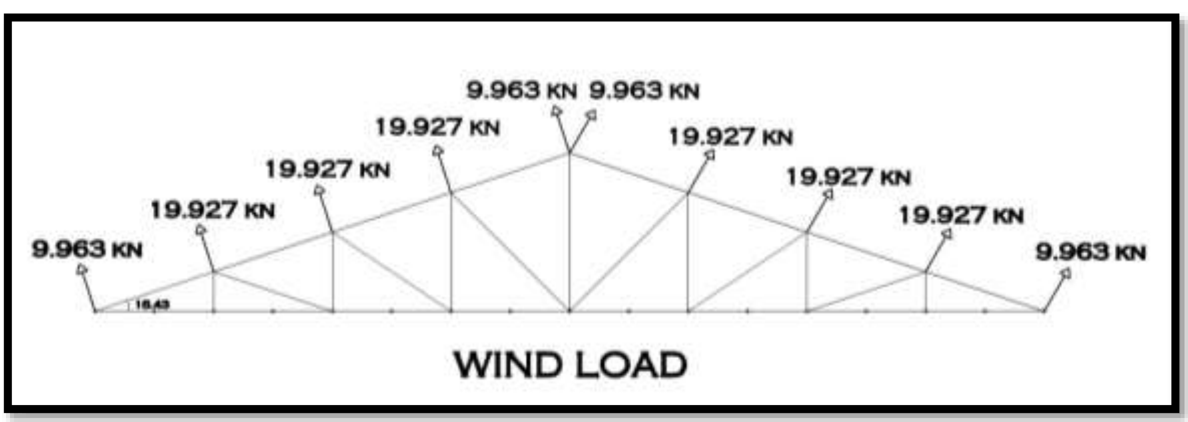


Fig 17. Wind Load on Each Panel Points.

CHAPTER 5

RESULT AND CONCLUSION

The Results with Stadd pro are shown below:-

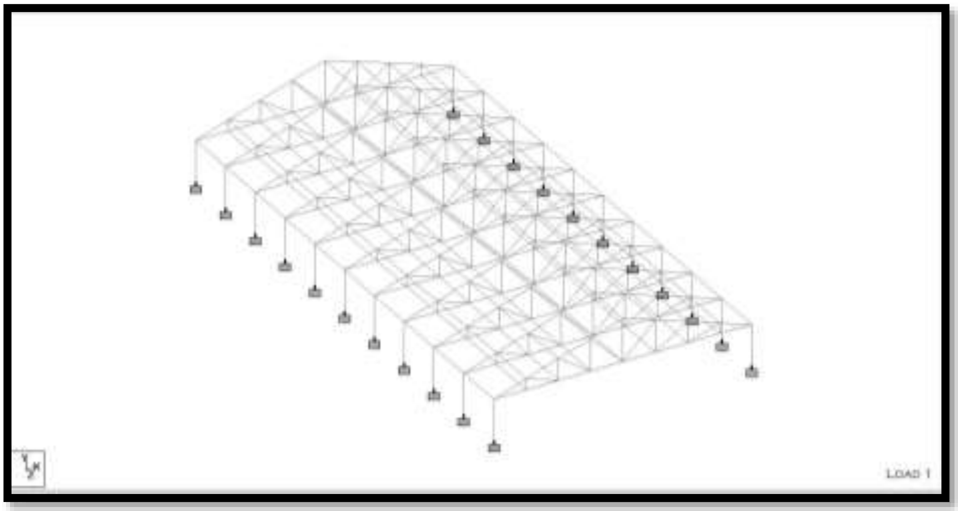


Fig 18. Model in Stadd pro.

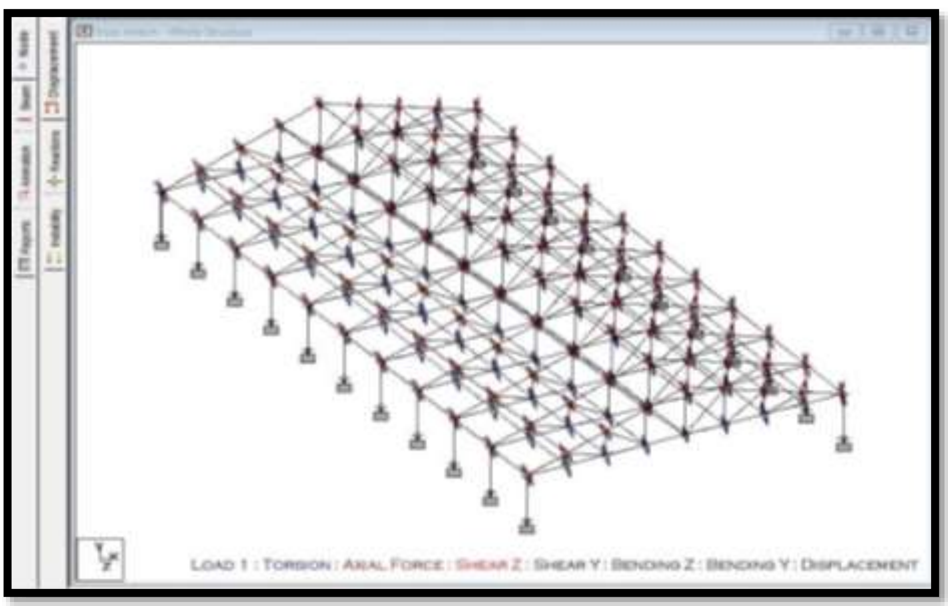
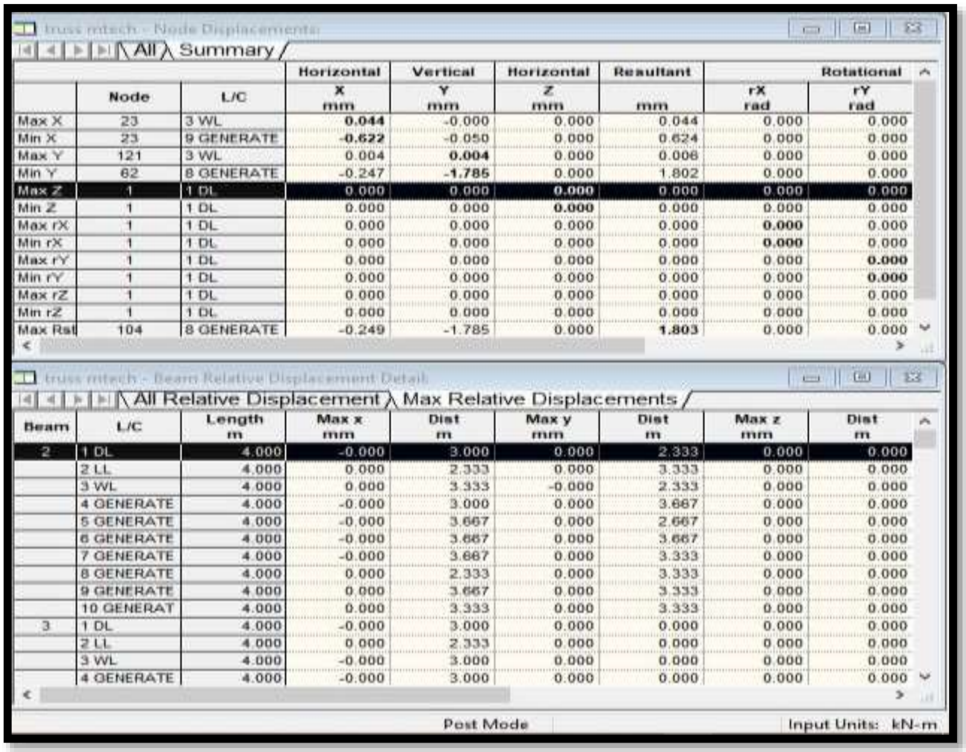


Fig 19. All Forces And Displacement At All Points



All | Summary

	Node	L/C	Horizontal X mm	Vertical Y mm	Horizontal Z mm	Resultant mm	Rotational rX rad	rY rad
Max X	23	3 WL	0.044	-0.000	0.000	0.044	0.000	0.000
Min X	23	9 GENERATE	-0.622	-0.050	0.000	0.624	0.000	0.000
Max Y	121	3 WL	0.004	0.004	0.000	0.006	0.000	0.000
Min Y	62	8 GENERATE	-0.247	-1.785	0.000	1.802	0.000	0.000
Max Z	1	1 DL	0.000	0.000	0.000	0.000	0.000	0.000
Min Z	1	1 DL	0.000	0.000	0.000	0.000	0.000	0.000
Max rX	1	1 DL	0.000	0.000	0.000	0.000	0.000	0.000
Min rX	1	1 DL	0.000	0.000	0.000	0.000	0.000	0.000
Max rY	1	1 DL	0.000	0.000	0.000	0.000	0.000	0.000
Min rY	1	1 DL	0.000	0.000	0.000	0.000	0.000	0.000
Max rZ	1	1 DL	0.000	0.000	0.000	0.000	0.000	0.000
Min rZ	1	1 DL	0.000	0.000	0.000	0.000	0.000	0.000
Max Rst	104	8 GENERATE	-0.249	-1.785	0.000	1.803	0.000	0.000

All Relative Displacement | Max Relative Displacements

Beam	L/C	Length m	Max x mm	Dist m	Max y mm	Dist m	Max z mm	Dist m
2	1 DL	4.000	-0.000	3.000	0.000	2.333	0.000	0.000
	2 LL	4.000	0.000	2.333	0.000	3.333	0.000	0.000
	3 WL	4.000	0.000	3.333	-0.000	2.333	0.000	0.000
	4 GENERATE	4.000	-0.000	3.000	0.000	3.667	0.000	0.000
	5 GENERATE	4.000	-0.000	3.667	0.000	2.667	0.000	0.000
	6 GENERATE	4.000	-0.000	3.667	0.000	3.667	0.000	0.000
	7 GENERATE	4.000	-0.000	3.667	0.000	3.333	0.000	0.000
	8 GENERATE	4.000	0.000	2.333	0.000	3.333	0.000	0.000
	9 GENERATE	4.000	0.000	3.667	0.000	3.333	0.000	0.000
	10 GENERAT	4.000	0.000	3.333	0.000	3.333	0.000	0.000
3	1 DL	4.000	-0.000	3.000	0.000	0.000	0.000	0.000
	2 LL	4.000	0.000	2.333	0.000	0.000	0.000	0.000
	3 WL	4.000	-0.000	3.000	0.000	0.000	0.000	0.000
	4 GENERATE	4.000	-0.000	3.000	0.000	0.000	0.000	0.000

Post Mode Input Units: kN-m

Fig 20. Summary of All loads Combination vs Displacements.

The maximum deflection according to software is 0.004 mm .As the deflection is under permissible limit thus the structure model is safe. L /25 = 25000/250 = 100 mm.

Various section details used in the structures are mention in given table:-

Sr. No	Types of Member	Member No.	Design section (Staad pro)
1	Main tie	1-8	ISWB600
2	Principal rafter	9-16	ISWB600
3	Vertical Ties	17,19,21,23,25,27,29	2 ISA 200 X 200 X 12
4	Inclined Ties	18,20,22,24,26,28	2 ISA 200 X 200 X12

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4. IS 875 PART 2
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10. BENTLEY STAAD PRO V8I.