JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY, WAKNAGHAT

Project Report submitted in fulfillment of the requirement for the degree of

Bachelor of Technology

in

Civil Engineering

on

Analysis & Design of Transfer Point: Coal Handling Plant

Under guidance of

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Asst. Professor

(Department of Civil Engineering)

By:-

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BHAWUK GARG 101693

MIKHIL KAUSHAL 101415

CERTIFICATE

This is to certify that the work titled "**Analysis and Design of Transfer Point:Coal Handeling Plant** " submitted by Akshat Agarwal, Bhawuk Garg and Mikhil Kaushal in fulfillment for the award of degree of B.Tech Civil Engineering of Jaypee University of Information technology, Waknaghat has been carried out under my supervision. This work has not been submitted partially or wholly to any other university or institute for the award of this or any other degree or diploma.

Mr. Mani Mohan Date: / /

Assistant Professor (Grade –II)

Department of Civil Engineering

ACKNOWLEDGMENT

We would like to express our deepest appreciation to our project guide Asst. Prof. **Mr. Mani Mohan** (Department. Of Civil Engineering) for his guidance for the duration of this project. His efforts and his guidelines were always an opportunity for us to learn and played an important role in the completion of this project.

We would also like to thank Prof. Dr. Veeresh Gali, Project Co-ordinator who was always present for comments and suggestions.

ABSTRACT

The purpose of this project is to be introduced to the design of steel structures using the Indian Standard Code IS 800:2007.This Indian Standard was adopted by Bureau of Indian Standard after the draft finalized by Structural Engineering Structural Sections Sectional Committee has been approved by Civil Engineering Division Council.IS 800 is the basic code for general construction in steel structures.

Anyone managing the construction process needs a basic understanding of the engineer's environment and the basic understanding of how a structure behaves. Constructors must be able to address a number of technical questions at the project site including structural issues that sometimes are not addressed by the design professionals. Since the safety of construction workers as well as the strength and stability of structures during the construction phase is of paramount importance, construction managers need this knowledge.

There are a variety of software programs which are available for the different specialized disciplines of civil engineering. Most civil engineers practice in specialized subsets of civil engineering, such as geotechnical engineering, structural engineering, transportation engineering, hydraulic engineering, environmental engineering, project and construction management. **STAAD or (STAAD.Pro)** is the structural engineering professional's choice for steel, concrete, timber, aluminum, and cold-formed steel design of virtually any structure including culverts, petrochemical plants, tunnels, bridges, piles, and much more through its flexible modeling environment, advanced features, and fluent data collaboration. STAAD.Pro allows structural engineers to analyze and design virtually any type of structure through its flexible modeling environment, advanced features and fluent data collaboration. Its wide use in the field of civil engineering makes it of an utter importance to learn.

Steel Design is done with limit state method conforming to IS-800:2007. The design results given by Staad Pro are compared with the results obtained manually. Ultimately, the various steel sections for beams and columns are obtained. After designing the building, the post processing mode in Staad Pro can be used to study the bending moment and shear force values with the generated diagrams. We can also check the deflections of the members under the given load.

Design and analysis of complicated and high-rise structures need very time taking and cumbersome calculations using conventional manual methods. STAAD.Pro provides us a fast, efficient, easy to use and accurate platform for analysing and designing structures. The design involves application of various loads (Dead, Live, Earthquake, Wind etc.) on the structure and analyzing the structure in the Staad Pro.

KEYWORDS: Bureau of Indian Standards, Limit State method, IS 800:2007, STAADPro.

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1. INTRODUCTION

1.1 About NTPC

India's largest power company, NTPC was set up in 1975 to accelerate power development in India. NTPC is emerging as a diversified power major with presence in the entire value chain of the power generation business. Apart from power generation, which is the mainstay of the company, NTPC has already ventured into consultancy, power trading, ash utilization and coal mining. NTPC ranked 384th in the '2013, Forbes Global 2000' ranking of the World's biggest companies. NTPC became a Maharatna company in May, 2010, one of the only four companies to be awarded this status.

The total installed capacity of the company is 42,454 MW (including JVs) with 16 coal based and 7 gas based stations, located across the country. In addition under JVs, 7 stations are coal based & another station uses naptha/LNG as fuel and 2 renewable energy projects. NTPC has been operating its plants at high efficiency levels. Although the company has 16% of the total national capacity, it contributes 25.6% of total power generation due to its focus on high efficiency.

1.2 Vidhyanchal Project

1) Location: Sidhi, Madhya Pradesh

2) Approved Capacity: 4760 MW (Stage-I 1260 MW + Stage-II 1000 MW + Stage-III 1000MW + Stage-IV 1000MW + Stage-V 500 MW).

3) Installed Capacity: 3760 MW

4) Coal Source: Nigahi Mines

5) Unit Sizes: Stage I: 6x 210 MW + Stage-II: 2x500 MW + Stage III: 2x 500 MW.

1.3 Structure & Design

This Project pertains to the structural design of Transfer Point TP-8A of CHP (Coal Handling Plant) package of NTPC's Vindhyachal Super Thermal power project stage-IV (2x500MW). The overall size of the building is 7m x 7m x 17.45m high. This TP is cladded on sides as per specification. The main floors and Roof are of RCC and other auxiliary floors/platforms are of chequered.plate.

This Structure is adjacent to the existing structure TP-8.The extension of TP-8 is done with some structural element from Upcoming structure TP-8A framing into the Existing Column C1, C2 and D2. The various Dead Load and Live Load acting at different points on beams are transferred to the nearest columns through the grid of beams. The lateral loads at various floors are transferred to end frames of the house through RCC slab or plan bracing as applicable. The end frames are braced structures, which in turn will transfer load to respective foundations. The design involves application of various loads (Dead, Live, Earthquake, Wind etc.) on the structure and analyzing the structure in the Staad Pro.

After analyzing for all the possible loading conditions, the results obtained by analyzing the frame will be compared. The loading condition in which resulting moments and axial forces are largest will be considered as the building should be able to sustain the worst possible situation. Finally, after obtaining the sections the amount of the steel required will be calculated and the cost estimation will be performed.

2. LITERATURE

2.1 Steel Definition

Gary S. Berman stated that steel is a common building material used throughout the construction industry. It forms the skeleton for the building or structure and basically holds everything together. Steel is widely used as a building material. It is because of its design simplicity, mechanical properties and ease and speed of construction. If there is any extension needed on a steel structure, the new structures can be just welded or bolted to the existing structure. And, still it will give the same strength. Steel has a variety of properties to suit different requirements which are strength, ductility, weld ability and corrosion resistance. Besides, steel has also a special feature. It will not break directly when it is loaded with excessive loading. It will buckle first, until it reaches its maximum capacity, then only it fails.

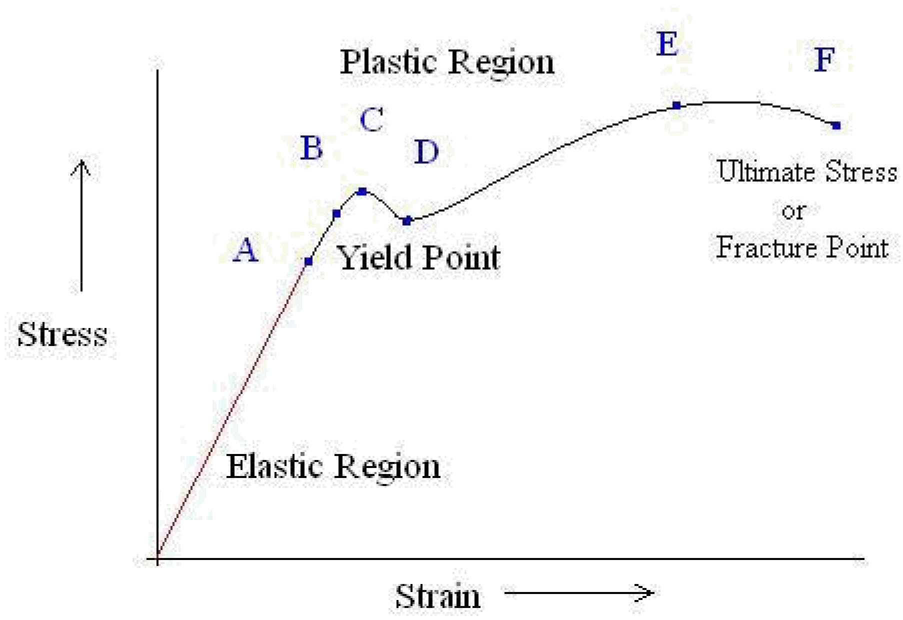

Fig 2.1. Stress Strain Curve of Steel

Steel will go through the yield point before it reaches the ultimate stress. Usually, the steel structure will be designed on its yield point. Reason being is to save cost since the steel structure is expensive. So, basically, the steel is stretched until it deforms to its yield point. Thus, the steel structure length is extended and the needs of more steel pieces can be reduced. Steel is shaped into several sections for the construction purposes which are I-section (Universal Beam), H-section (Universal Column), circular hollow section (CHS), rectangular hollow section (RHS), square hollow section (SHS), unequal angles, equal angles, double angles and many other shapes.

This is why steel is preferred to be used in the construction as compared of concrete and timber. There are many sections available in the market. The engineer only needs to choose which design suits his design requirements. Other than that, this might cut off the cost of construction as well. The chosen design unquestionably satisfies the building requirements.

2.2 Steel Structure Elements

2.2.1 Beam

A structural beam is a component used in construction to add strength to any structure or design. Manufactured of steel, concrete or wood, the structural beam is typically used to span an open element of a structure, as well as to give support underneath a very heavy component of a structure. I beam (Universal Beam) is the most common type of beam used. Concrete structural beam manufacture often involves a steel I beam as the reinforcement in concrete for use in building bridges, buildings, and other concrete structures. Besides, channel section and angle are sometimes used also for the beam. Beside concrete and steel, beam can be made of plastic and wood.

Below are the common sections that are used for the beam design.

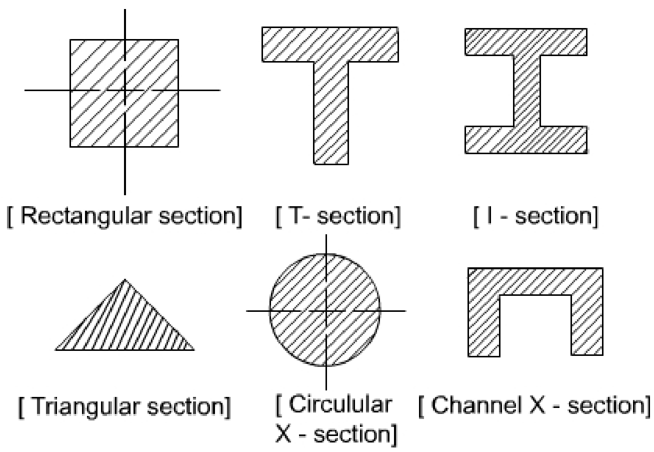

Fig 2.2. Various section of Steel

On the construction, there are some combinations of beam supports that can be installed. Different combination of the supports, the response of the beam towards the applied load would be different as well. The most common combinations used are cantilever beam (fixed – free) and simply-supported beam (pin – roller or pin – pin). Problem that usually beam has is bending. Why bending? Because it is loaded with lateral loading. Therefore, if it is observed from the cross sectional area of the beam (assumed the loading is imposed from the top), the top part of the beam will experience axial compression, whereas the bottom part will experience axial tension.

On this project, the beam section used is Universal Beam (I-Beam). Since the beams are all primary beams, therefore, they have to be checked for its web bearing and web buckling. Most importantly, the shear buckling, shear capacity, moment capacity and allowable deflection must be checked first before assigning a section. This is to ensure the safety of the building constructed.

2.2.2 Column

A structural member loaded axially in compression is generally called a compression member. Vertical compression members in buildings are called columns, posts or stanchions. A compression member in roof trusses is called struts and in a crane is called a boom.

Column is a vertical structural member that transmits the load from ceiling/ roof slab and beam, including its self-weight to the foundation. Columns are normally subjected to a pure compressive load. The most common used columns are RCC (Reinforced Concrete) columns.

Columns which are short are subjected to crushing and behave like members under pure compression. Columns which are long tend to buckle out of the plane of the load axis.

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Caprani mentioned two main parameters governing column design.

- Bracing: if the column can sway, additional moments are generated through the $P - \delta$ effect. This does not affect braced column.
- Slenderness ratio: The effective length divided by the lateral dimension of the column. Low values indicate a crushing failure, while high values denote buckling.

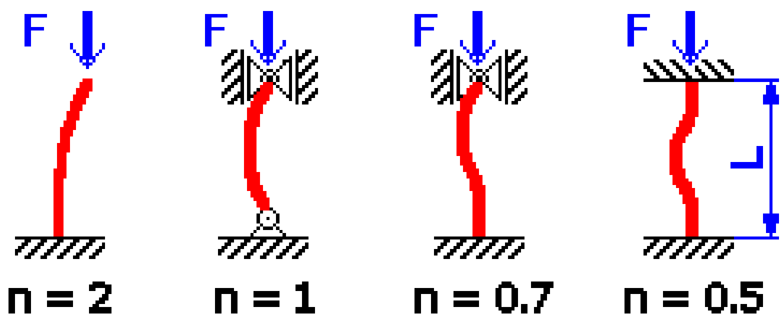


Fig 2.3. Effective Length of Different Supports Combination of Column

The steel column can also fail if the design is not done properly. A long compression member may fail due to buckling stress whereas the short compression member may fail due to yielding of material. Buckling of a column may occur even the maximum stresses in the material are less than the yield point of the material. Buckling means lateral deflection of the column.

Effective Length of Compression Member

Table below gives the values of effective length recommended by the Indian Standard, IS 800. The actual length L of the compression member should be taken as the length from centre-to-centre of intersection of supporting members or the cantilevered length in the case of free standing struts.

Table: Equivalent length for various end conditions

	Type	**Effective length of member l**
1	Effectively held in position and restrained in direction at both ends.	0.67 L
2	Effectively held in position at both ends restrained in direction at one end.	0.85 L
3	Effectively held in position at both ends but not restrained in direction.	L
4	Effectively held in position and restrained in direction at one end and at the other end effectively restrained in direction but not held in position.	L
5	Effectively held in position and restrained in direction at one end and the other end partially restrained in direction but not held in position.	1.5 L
6	Effectively held in position and restrained in direction at one end but not held in position or restrained in direction at the other end.	2.0 L

Note:

1. L is the unsupported length of compression member.
2. For battened struts, the effective length should be increased by 10%.

On this project, the column is designed by using Universal Beam instead of Universal Column. Universal Beam will be more vulnerable to buckle as compared of Universal Column. Universal Column has approximately the same magnitude of flange width and web length, whereas, the Universal Beam has the web length greater than the flange width. Therefore, web buckling might happen on UB column. Nevertheless, if the design is done properly according to the specification, it is hoped the section will not show any sign of failure.

Maximum Slenderness Ratio:

According to Indian Standard IS 800, the slenderness ratio should not exceed the values given in the table below:

No.	Type of Member	Slenderness ratio $\lambda = \frac{l}{r}$
1	A member carrying compressive loads resulting from dead and superimposed loads.	180
2	A member subjected to compressive loads resulting from wind/earthquake forces provided the deformation of such members does not adversely affect the stress in any part of the structure.	250
3	A member normally carrying tension but subjected to reversal of stress due to wind or earthquake forces.	350
4	Tension member (other than pre-tensioned member)	400

2.2.3 Bracing

Bracing in the form of diagonal structural sections is provided to frames to prevent, or at least to restrict, sway in single and multi-storey buildings . For a frame to be classified as braced, it must possess a bracing system which is adequately stiff. The frame without the bracing system can be treated as fully supported laterally and as having to resist the action of the vertical loads only. The bracing system resists all the horizontal loads applied to the frames it braces, any vertical loads applied to the bracing system and the effects of the initial sway imperfections from the frames it braces and from the bracing system itself.

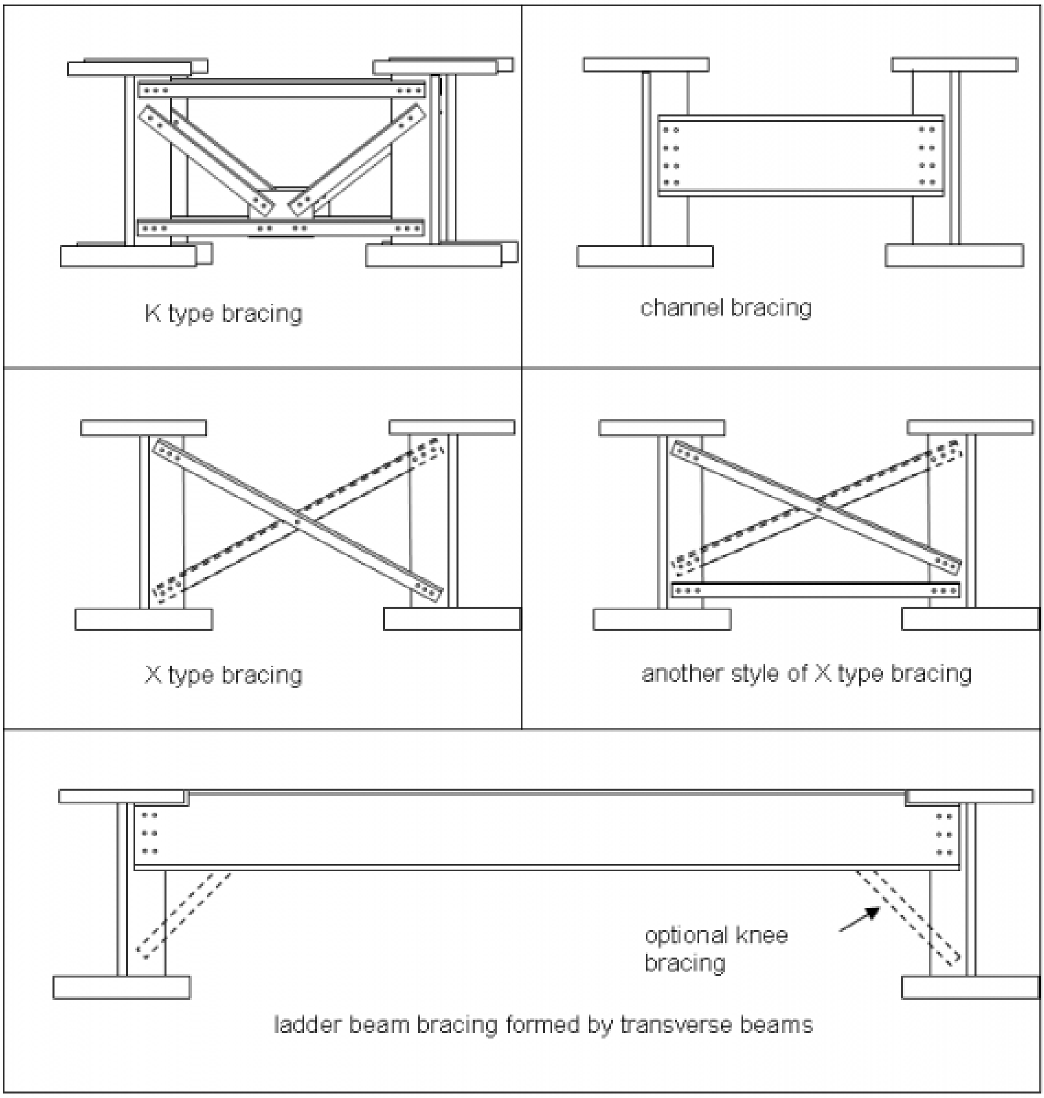


Fig 2.4. Various types of bracings

In a *Braced Frame* building, the resistance to horizontal forces is provided by two orthogonal bracing systems:

- Vertical bracing. Bracing in vertical planes (between lines of columns) provides load paths to transfer horizontal forces to ground level and provide a stiff resistance against overall sway.
- Horizontal bracing. At each floor level, bracing in a horizontal plane, generally provided by floor plate action, provides a load path to transfer the horizontal forces (mainly from the perimeter columns, due to wind pressure on the cladding) to the planes of vertical bracing.

Bracing types available for incorporation into the structural system range from concentric simple *K or X brace* between two columns to knee bracing and eccentric bracings with complicated geometry requiring computer solutions.

In *eccentric bracing* system the connection of the diagonal brace is deliberately offset from the connection between beam and vertical column.

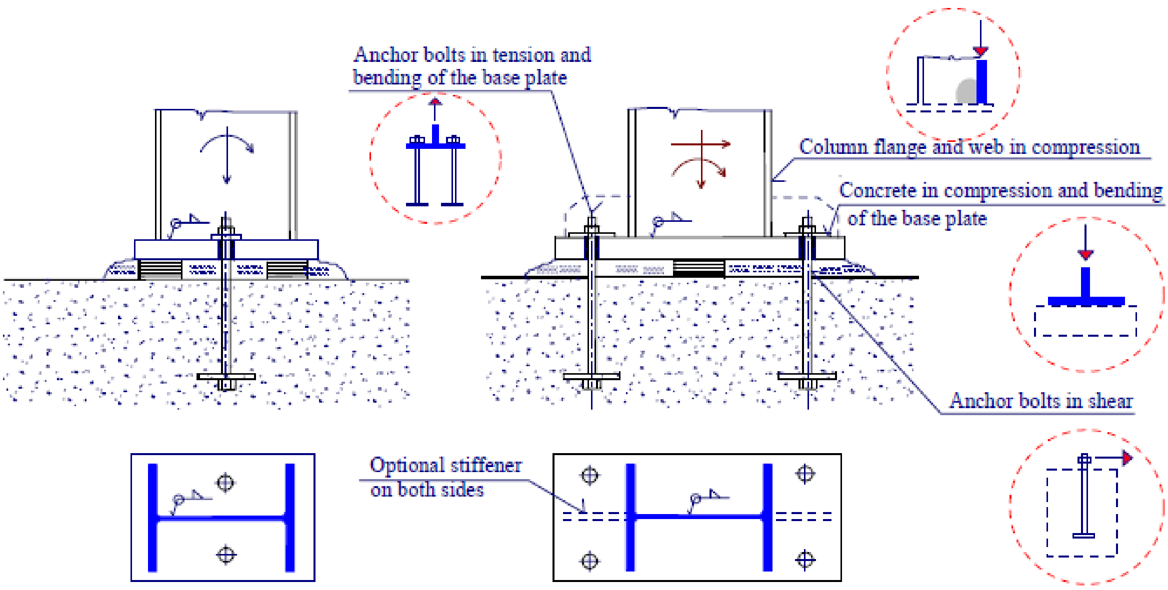
The *K-form* is common since the bracings do not participate extensively in carrying gravity load, and can be designed for axial forces due to wind without gravity axial forces.

In the *X-form* and single brace forms, gravity axial forces may dominate in the design of braces.

The building sway under wind is significantly reduced by the introduction of these trusses.

2.2.4 Base Plate

In general they are designed with unstiffened base plates, but stiffened base plates may be used where the connection is required to transfer high bending moments. The column base is usually supported by either a concrete slab or a sub-structure. The traditional approach for the design of pinned bases results in a base plate thickness of sufficient stiffness to ensure a uniform stress under the base plate and therefore the base plate can be modelled as a rigid plate.



a) Bolt inside the base plate   b) Bolt outside the base plate

Fig 2.5. Typical Column base assembly and selection of components

The traditional design of moment-resisting column bases involves an elastic analysis based on the assumption that the sections remain plane. By solving equilibrium equations, the maximum stress in the concrete foundation (based on linear stress distribution) and the tension in the holding down assembly may be determined. Whilst this procedure has proved satisfactory in service over many years, the approach ignores the flexibility of the base plate in bending (even when it is strengthened by stiffeners), the holding down assemblies and the concrete. The concept, transfers the flexible base plate into an effective rigid plate and allows stress in the concrete foundation equal to the resistance in concentrated compression. A plastic distribution of the internal forces is used for calculations at the ultimate limit state

2.2.5 Monorail

The term **monorail** or **industrial monorail** is used to describe any number of transport systems in which a chair or carrier is suspended from, or rides on, an overhead rail structure. Unlike the well-known duo-rail system, there are many rail-guided transport options which have been described as monorails, so that tracing the history presents ademarcation problem regarding what should be included and what should be omitted.

The overhead traveling crane, hoist, and monorail system industry includes a diverse assortment of products that fit within a narrowly defined segment of the materials handling equipment industry. Not to be confused with various types of mobile cranes used in construction projects, overhead cranes are variously structured machines that "travel" along a runway structure or pair of tracks located above the work floor of a plant or factory. They are further characterized by the presence of a fixed or trolley-mounted hoisting system that is connected to the tracks by a bridge structure, which consists of either a single or double girder.

The industry manufactures three basic kinds of overhead traveling cranes that accommodate the vast majority of materials handling needs. The first type is the overhead bridge crane, which is fixed to an overhead beam running the length of the building. Generally regarded as the most rugged of all overhead traveling cranes, this class of crane is noted for its ability to cover the entire width and length of a plant. The jib crane is the second variety of crane produced by the industry. It is usually mounted to a wall or pillar and is used to service a smaller area of a plant, usually the area of a single workstation. Gantry cranes, which are mounted overhead and are able to service a particular bay or workstation, comprise the third category.

2.2.6 Cantilever Retaining Wall

- Cantilever wall are usually of reinforced concrete and work on the principles of leverage.
- Have much thinner stem, and utilize the weight of the backfill soil to provide most of the resistance to sliding and overturning.
- Most common type of earth- retaining structure.
- The cantilever retaining wall (“cantilever wall”) constructed of reinforced Portland-cement concrete (PCC) was the predominant type of rigid retaining wall used from about the 1920s to the 1970s
- Earth slopes and earth retaining structures are used to maintain two different ground surface elevations.

FUNCTION

To retain the soil at a slope that is greater than it would naturally assume, usually at a vertical or near vertical position.

Earth Pressure (P)

Earth pressure is the pressure exerted by the retaining material on the retaining wall. This pressure tends to deflect the wall outward. There are two types of earth pressure and they are; Active earth pressure or earth pressure (P_a) and Passive earth pressure (P_P). Active earth pressure tends to deflect the wall away from the backfill. Earth pressure depends on type of backfill, the height of wall and the soil conditions

2.3 Transfer Point

A *Transfer Point* is a location on a conveyor where the material is loaded or unloaded. A typical *Transfer Point* is composed of metal chutes that guide the flow of material. The design of *Transfer Point* will greatly affect the life of components as well as maintenance costs and safety.

The huge amount of coal is usually supplied through railways. A railway siding line is taken into the power station and the coal is delivered in the storage yard. The coal is unloaded from the point of delivery by means of wagon tippler. It is rack and pinion type. The coal is taken from the unloading site to dead storage by belt conveyors. The belt deliver the coal to 0m level to the pent house and further moves to transfer point 8.

The transfer points are used to transfer coal to the next belt. The belt elevates the coal to breaker house. It consists of a rotary machine, which rotates the coal and separates the light dust from it through the action of gravity and transfer this dust to reject bin house through belt.

The belt further elevates the coal to the transfer point 7 and it reaches the crusher through belt. In the crusher a high-speed 3-phase induction motor is used to crush the coal to a size of 50mm so as to be suitable for milling system. Coal rises from crusher house and reaches the dead storage by passing through transfer point 8.

2.4 About Structure

There are two types of structures i.e. ***Rigid Frame*** and ***Braced Frames***. A *Rigid frame* structure is a structure made up of linear elements, typically beams and columns that are connected to one another at their ends with joints that do not allow relative rotations to occur between the ends of attached members, although the joints themselves may rotate as a unit. Members are essentially continuous through the joints. As with continuous beams, rigid-frame structures are statically indeterminate.

Many *rigid-frame* structures resemble simpler post-and-beam systems in appearance, but radically different in structural behavior, owing to joint rigidity, which can be sufficient to enable a framed structure to carry significant lateral loads.

2.5 STAAD.Pro V8i Review

STAAD in STAADPro stands for Structural Analysis and Design. It is the most well-known engineering structural design software. According to “STAAD.Pro V8i” ,STAAD.Pro is the structural engineering professional’s choice for steel, concrete, timber, aluminium and cold-formed steel design of virtually any structure though its flexible modelling environment, advanced features, and fluent data collaboration.

STAAD.Pro is a comprehensive integrated FEA (Finite Element Analysis) and design solution, including a state of the art used interface, visualization tools and integrated codes. It is able to analyse a structure exposed to dynamic response, soil structure interaction of wind, earthquake and moving loads.

2.6 Cost Estimation

A cost estimate is the approximation of the cost of a program, project, or operation. The cost estimate is the product of the cost estimating process. The cost estimate has a single total value and may have identifiable component values. A problem with a cost overrun can be avoided with a credible, reliable, and accurate cost estimate. An estimator is the professional who prepares cost estimates. There are different types of estimators, whose title may be preceded by a modifier, such as building estimator, or electrical estimator, or chief estimator. Other professional titles may also prepare estimates or contribute to estimates, such as quantity surveyors, cost engineers, etc.

In a world of limited funds, as a project manage you're constantly deciding how to get the most return for your investment. The more accurate your estimate of project cost is, the better able you will be to manage your project’s budget. Therefore, estimating a project’s costs is important for several reasons:

- It enables you to weigh anticipated benefits against anticipated costs to see whether the project makes sense.
- It allows you to see whether the necessary funds are available to support the project.
- It serves as a guideline to help ensure that you have sufficient funds to complete the project.

Although you may not develop and monitor detailed budgets for all your projects, knowing how to work with project costs can make you a better project manager and increase your chances of project success.

A project budget is a detailed, time-phased estimate of all resource costs for your project. You typically develop a budget in stages — from an initial rough estimate to a detailed estimate to a completed, approved project budget. On occasion, you may even revise your approved budget while your project is in progress. Your project’s budget includes both direct and indirect costs.

Direct costs include the following:

- Salaries for team members on your project
- Specific materials, supplies, and equipment for your project
- Travel to perform work on your project
- Subcontracts that provide support exclusively to your project

Indirect costs fall into the following two categories:

Overhead costs: Costs for products and services for your project that are difficult to subdivide and allocate directly. Examples include employee benefits, office space rent, general supplies, and the costs of furniture, fixtures, and equipment.

You need an office to work on your project activities, and office space costs money. However, your organization has an annual lease for office space, the space has many individual offices and work areas, and people work on numerous projects throughout the year. Because you have no clear records that specify the dollar amount of the total rent that's just for the time you spend in your office working on just this project's activities, your office space is treated as an indirect project cost.

General and administrative costs: Expenditures that keep your organization operational (if your organization doesn't exist, you can't perform your project). Examples include salaries of your contracts department, finance department, and top management as well as fees for general accounting and legal services.

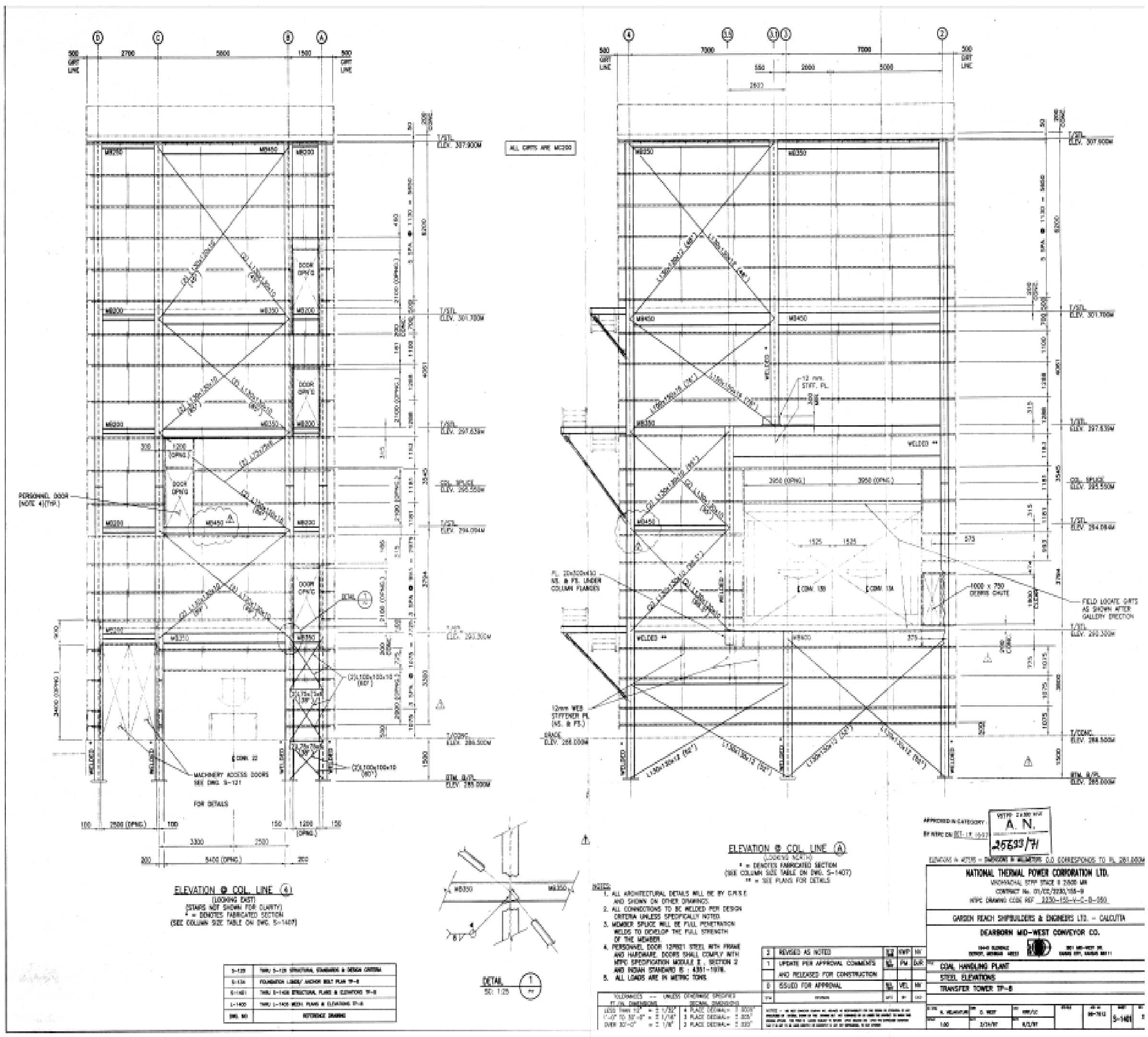
Our objective is to calculate the quantity and hence cost of the materials used in the construction of the structure.

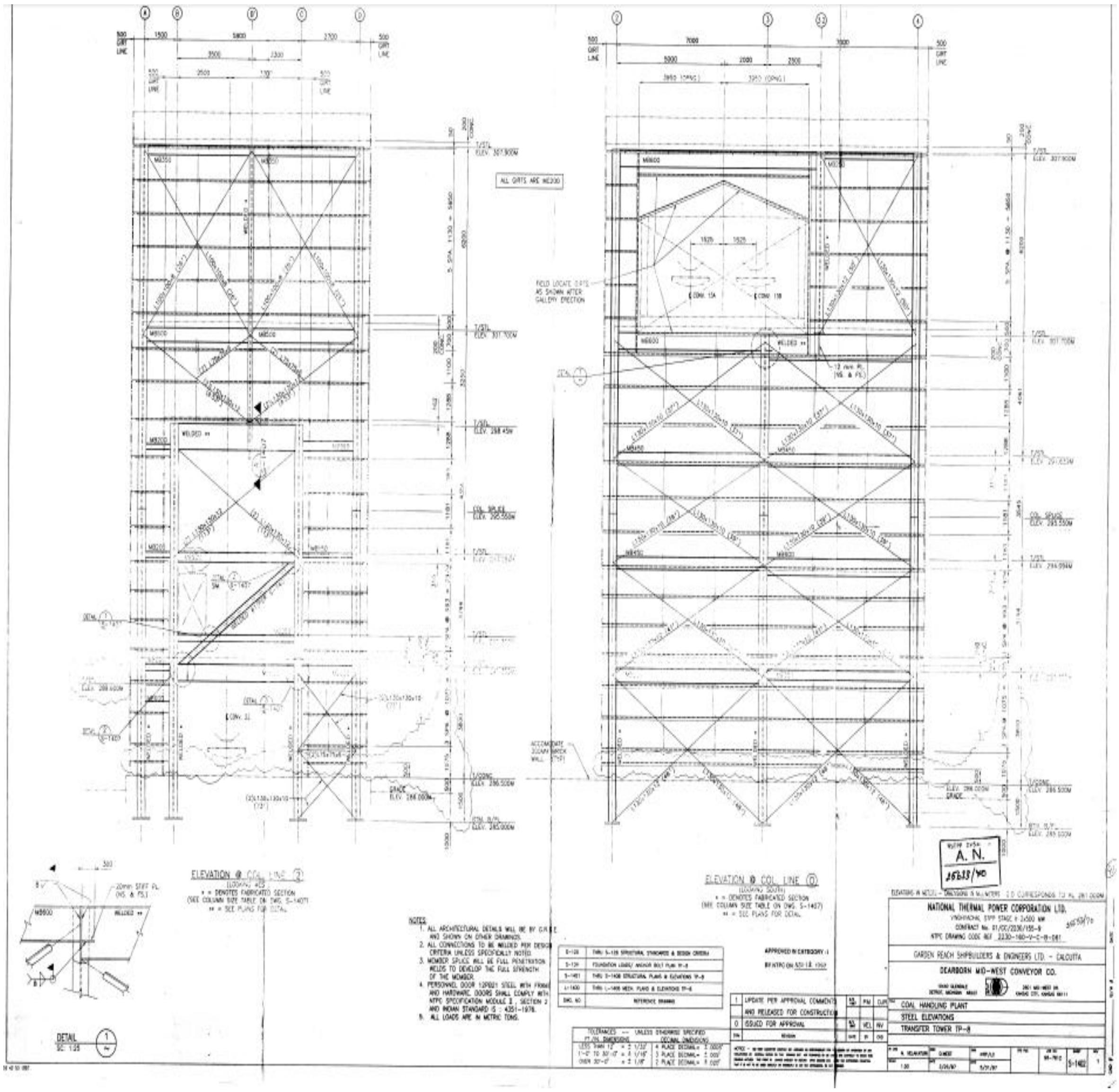
3. MATERIALS AND METHODS

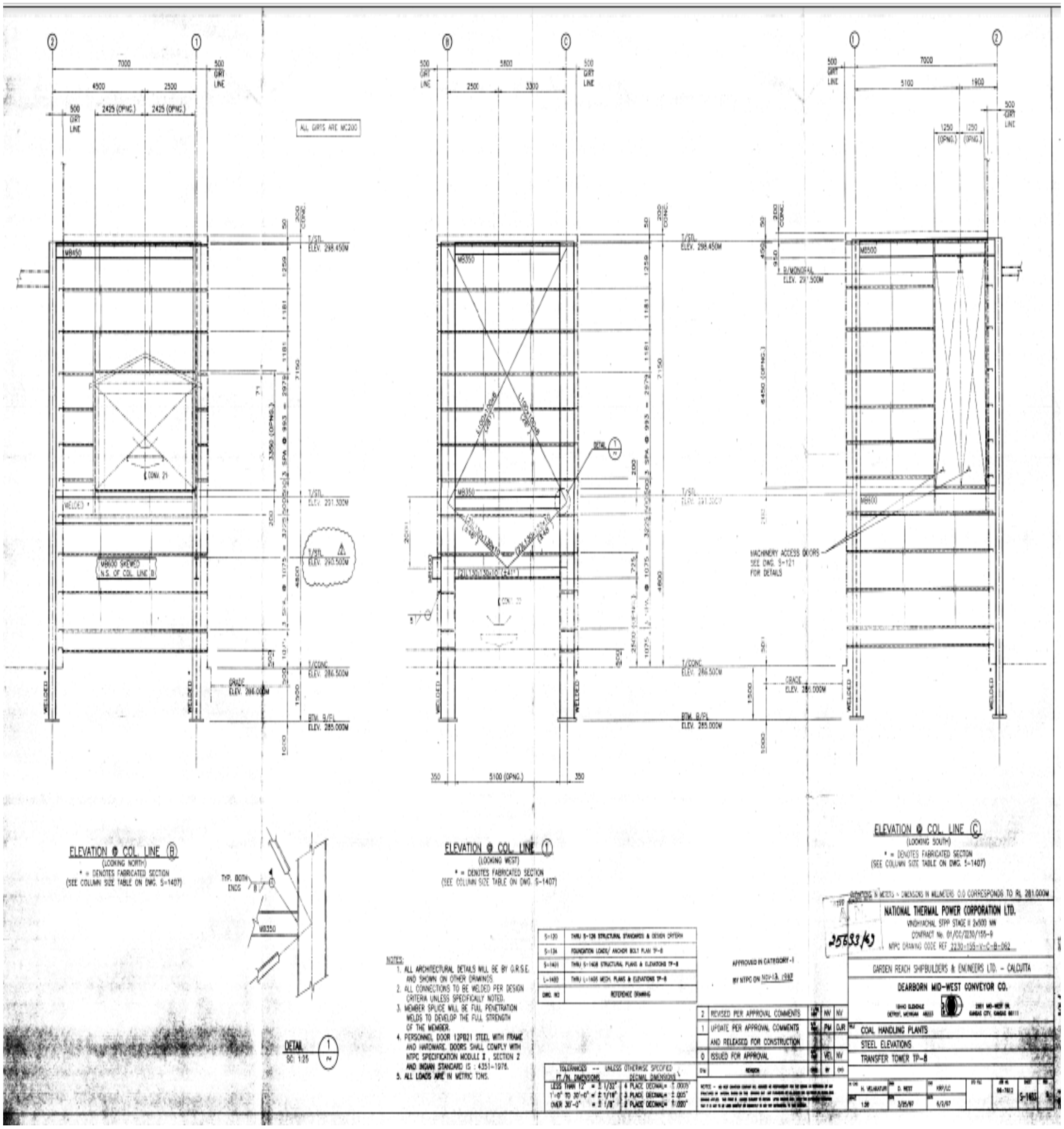
In this project we are using STAAD.pro as designing and analysis tool.

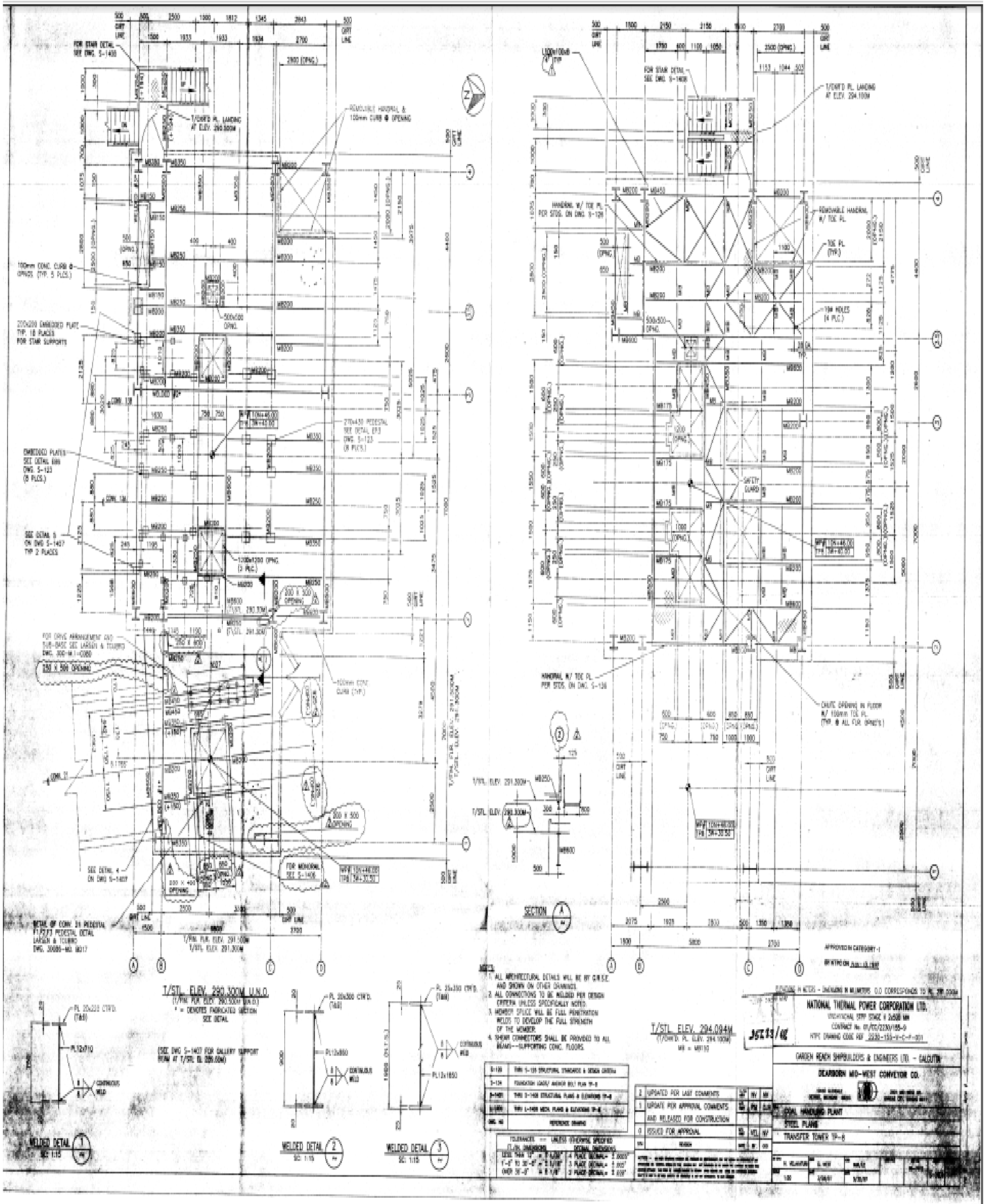
- Indian Standard Code 800:2007 is used as a reference tool for designing of steel structure.
- Indian Standard Code 875(Part-3) is used for the calculation of the wind loads considering that particular wind zone in which our structure lies.
- NTPC Specifications are being provided for the calculation of various loads.
- Design specifications for various elements i.e. of beams, angles etc. are also provided in the detailed project report by NTPC.

3.1 Plans and Elevations given by NTPC are as under:

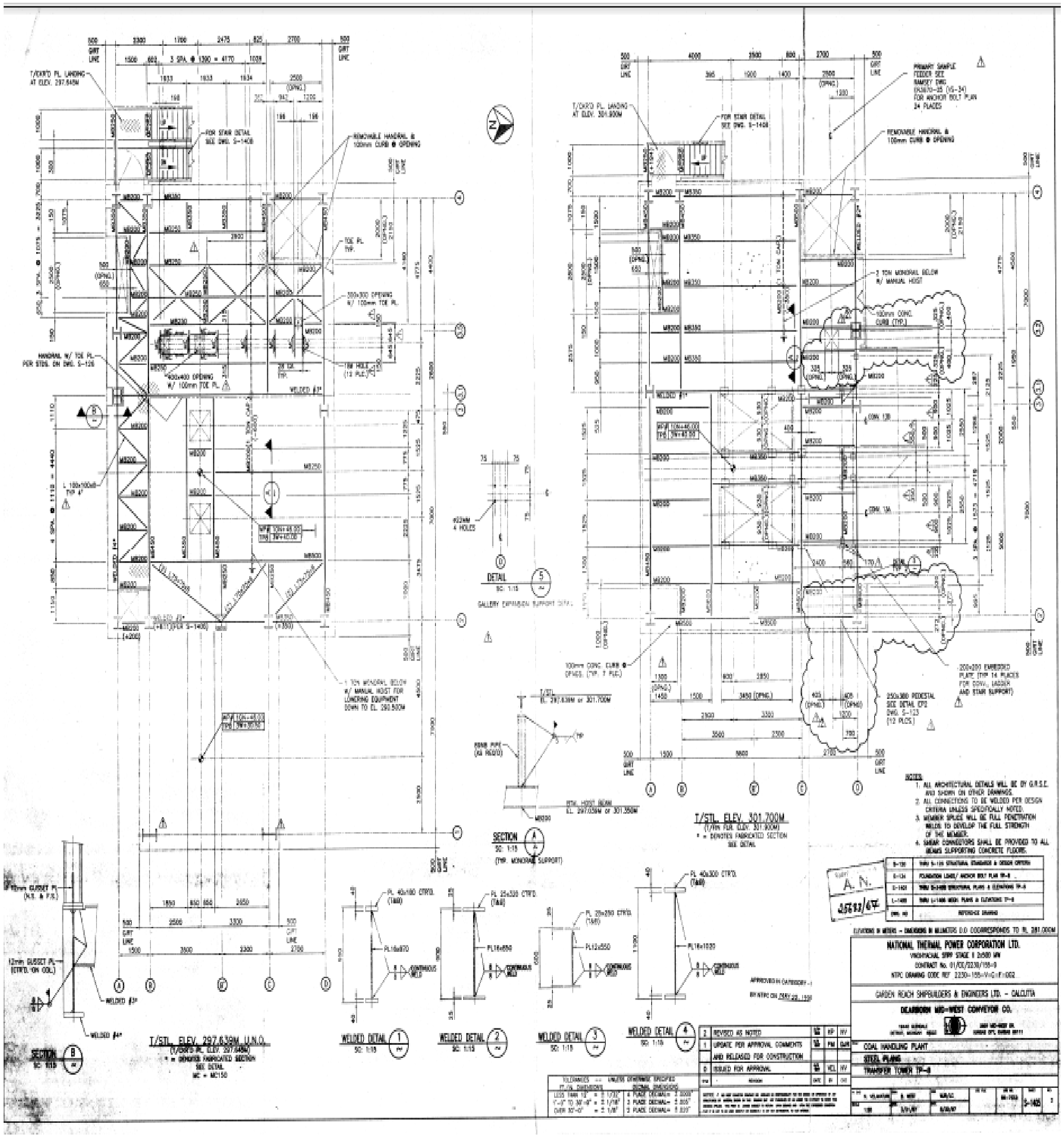

ELEVATION @ COL. LINE (4)
(LOOKING EAST)
(STAIRS NOT SHOWN FOR CLARITY)
ELEVATION @ COL. LINE (A)
(LOOKING NORTH)
ALL GIRTS ARE MC200
DETAIL 1
NATIONAL THERMAL POWER CORPORATION LTD.
GARDEN REACH SHIPBUILDERS & ENGINEERS LTD. – CALCUTTA
DEARBORN MID-WEST CONVEYOR CO.
COAL HANDLING PLANT
STEEL ELEVATIONS
TRANSFER TOWER TP-8


ELEVATION @ COL. LINE ②
ELEVATION @ COL. LINE ⓪
NATIONAL THERMAL POWER CORPORATION LTD.
GARDEN REACH SHIPBUILDERS & ENGINEERS LTD. - CALCUTTA
DEARBORN MID-WEST CONVEYOR CO.
COAL HANDLING PLANT
STEEL ELEVATIONS
TRANSFER TOWER TP-8
A. N.


ELEVATION @ COL. LINE (B)
(LOOKING NORTH)
* = DENOTES FABRICATED SECTION
(SEE COLUMN SIZE TABLE ON DWG. S-1407)
ELEVATION @ COL. LINE (1)
(LOOKING WEST)
* = DENOTES FABRICATED SECTION
(SEE COLUMN SIZE TABLE ON DWG. S-1407)
ELEVATION @ COL. LINE (C)
(LOOKING SOUTH)
* = DENOTES FABRICATED SECTION
(SEE COLUMN SIZE TABLE ON DWG. S-1407)
ALL GIRTS ARE MC200
MACHINERY ACCESS DOORS
SEE DWG. S-121
FOR DETAILS
NOTES:
1. ALL ARCHITECTURAL DETAILS WILL BE BY G.R.S.E. AND SHOWN ON OTHER DRAWINGS.
2. ALL CONNECTIONS TO BE WELDED PER DESIGN CRITERIA UNLESS SPECIFICALLY NOTED.
3. MEMBER SPLICE WILL BE FULL PENETRATION WELDS TO DEVELOP THE FULL STRENGTH OF THE MEMBER.
4. PERSONNEL DOOR 13PB21 STEEL WITH FRAME AND HARDWARE. DOORS SHALL COMPLY WITH NTPC SPECIFICATION MODULE I, SECTION 2 AND INDIAN STANDARD IS : 4351-1976.
5. ALL LOADS ARE IN METRIC TONS.
DETAIL (1)
SC: 1:25
NATIONAL THERMAL POWER CORPORATION LTD.
VINDHYACHAL STPP STAGE II 2x500 MW
CONTRACT No. 01/CC/2230/155-9
GARDEN REACH SHIPBUILDERS & ENGINEERS LTD. - CALCUTTA
DEARBORN MID-WEST CONVEYOR CO.
COAL HANDLING PLANTS
STEEL ELEVATIONS
TRANSFER TOWER TP-8
S-1405



SECTION A
T/STL. ELEV. 290.300M U.N.O.
T/STL. ELEV. 294.094M
WELDED DETAIL 1
WELDED DETAIL 2
WELDED DETAIL 3
NATIONAL THERMAL POWER CORPORATION LTD.
GARDEN REACH SHIPBUILDERS & ENGINEERS LTD. - CALCUTTA
DEARBORN MID-WEST CONVEYOR CO.
COAL HANDLING PLANT
STEEL PLANS
TRANSFER TOWER TP-8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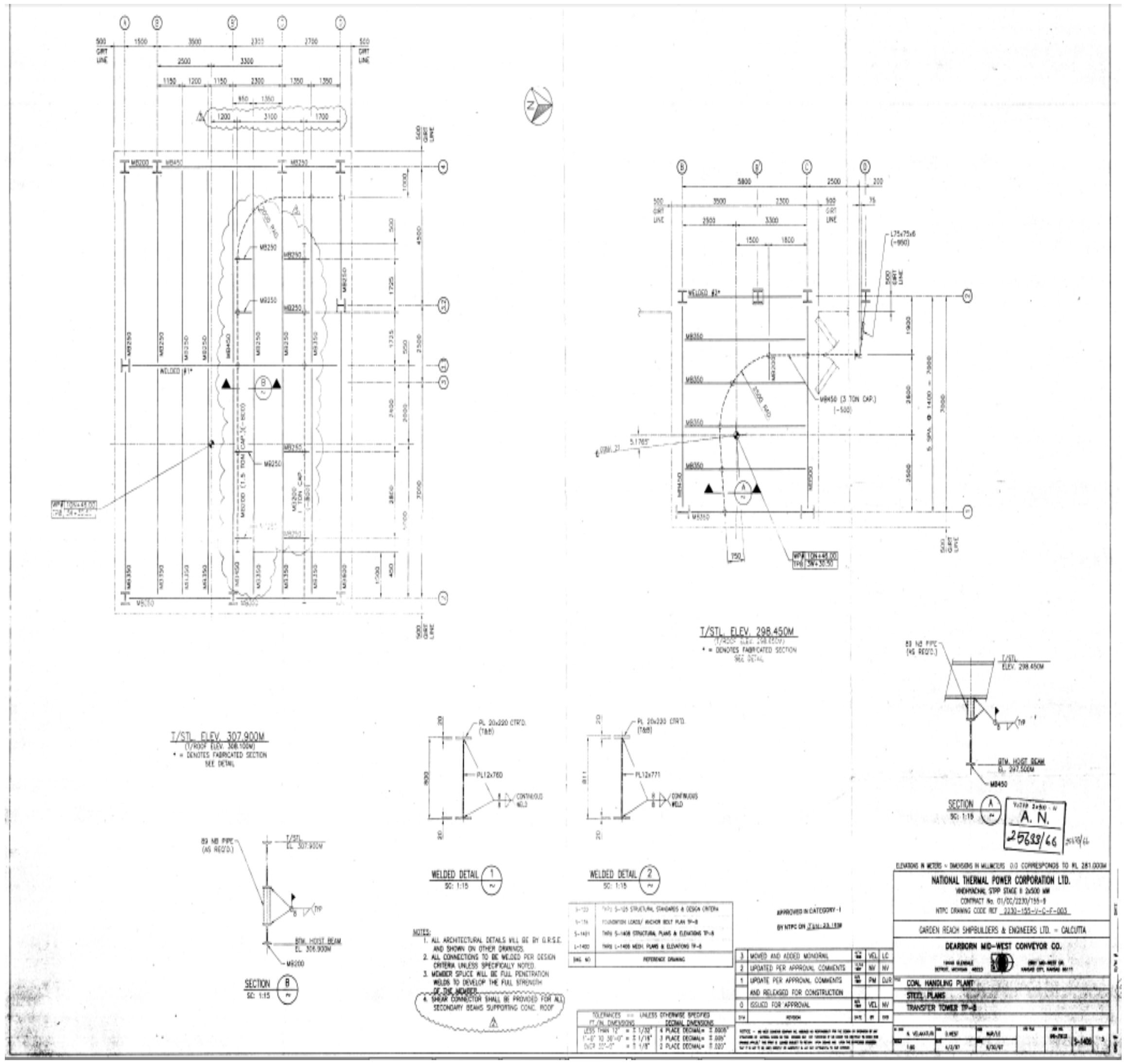
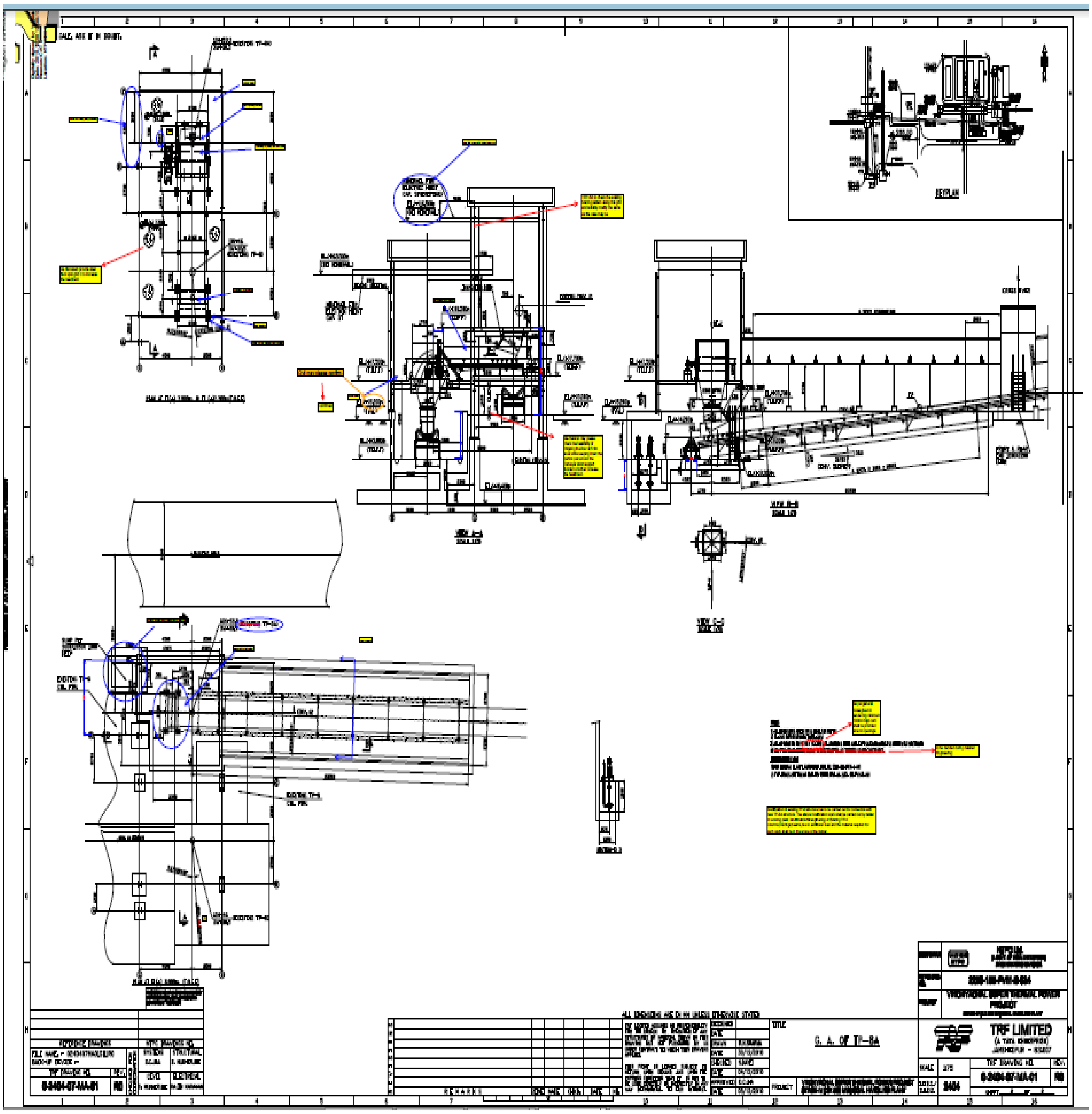
GARDEN REACH SHIPBUILDERS & ENGINEERS LTD. – CALCUTTA
DEARBORN MID-WEST CONVEYOR CO.
COAL HANDLING PLANT
STEEL PLANS
TRANSFER TOWER TP-8
T/STL. ELEV. 301.700M
T/STL. ELEV. 297.639M U.N.O.
SECTION A
SECTION B
DETAIL 5
WELDED DETAIL 1
WELDED DETAIL 2
WELDED DETAIL 3
WELDED DETAIL 4
NOTES:
1. ALL ARCHITECTURAL DETAILS WILL BE BY G.R.S.E. AND SHOWN ON OTHER DRAWINGS.
2. ALL CONNECTIONS TO BE WELDED PER DESIGN CRITERIA UNLESS SPECIFICALLY NOTED.
3. MEMBER SPLICE WILL BE FULL PENETRATION WELDS TO DEVELOP THE FULL STRENGTH OF THE MEMBER.
4. SHEAR CONNECTORS SHALL BE PROVIDED TO ALL BEAMS SUPPORTING CONCRETE FLOORS.
REVISED AS NOTED
UPDATE PER APPROVAL COMMENTS
AND RELEASED FOR CONSTRUCTION
ISSUED FOR APPROVAL
REFERENCE DRAWING


NATIONAL THERMAL POWER CORPORATION LTD.
GARDEN REACH SHIPBUILDERS & ENGINEERS LTD. - CALCUTTA
DEARBORN MID-WEST CONVEYOR CO.
COAL HANDLING PLANT
STEEL PLANS
TRANSFER TOWER TP-8
T/STL ELEV. 307.900M
T/STL ELEV. 298.450M
WELDED DETAIL 1
WELDED DETAIL 2
SECTION A
SECTION B


KEYPLAN
G. A. OF TP-8A
TRF LIMITED
(A TATA ENTERPRISE)
ALL DIMENSIONS ARE IN MM UNLESS OTHERWISE STATED
REMARKS

3.2 Methodology

The complete building is first modelled as 3D frame for existing structure then force on existing column which will aid TP-8A for supporting various structural frames for proposed structure is transferred to the model of TP-8A. The Dead Load and Live Load is of existing Structure is assumed as 600kg/m² and 500kg/m² respectively.

The Extension portion is been analyzed/designed for worst combination of loads through STAAD.pro and input and output files are enclosed in Annexure which are self-explanatory. From the STAAD output, the support reactions for various combinations are to be tabulated and the base plate, shear keys and foundation bolts are designed accordingly. The Existing Col. of TP 8 is also checked for its margin of safety.

Following are the design steps used for manual designing.

3.2.1 Design of Base Plate

1. Taking the M25 grade concrete, bearing strength of concrete is determined by 0.45fck.
2. The area of a slab base may be computed by:-

$$A = \frac{\mathrm{P}}{\text{bearing strength of concrete}}$$

Where A = required area of the base plate in mm^2

P = factored load

3. A square base plate is generally provided. The side of the base plate may be worked out by eq.

(D + 2b) x (bf + 2a) = A

Where, a = bigger projection of base plate beyond column in mm

b = smaller projection of base plate beyond column in mm

D= depth of column section mm

bf = width of the flange of column in mm

4. The intensity of pressure w, from the concrete pedestal is determined by

$$\mathrm{w} = \frac{P}{\mathrm{A}_1}$$

Where, w = intensity of pressure from concrete under the slab base in $\mathrm{N/mm}^2$

A_1 = area of base plate provide in mm^2

5. The minimum thickness t is calculated from equation

$$t = \sqrt{2.5w(a^2 - 0.3b^2)\frac{\gamma_{mo}}{f_y}}$$

Where, t should not be less than column flange thickness.

6. Moment in the cantilever part of the plate is calculated. Thickness is checked for the moment calculated from the equation

$$t = \sqrt{\frac{6M}{R}}$$

Where, M = moment in the cantilever part of slab

R =

7. Holding down bolts 2 or 4 in number and of 20 mm diameter are usually provided.
8. Bolts are checked under tension and shear.

3.2.2 Design of Cladding

1. A channel section is assumed, say MC150, dead load is calculated including self-weight, weight of sheeting and load on rail.
2. Wind loads are calculated as per IS 875(part3).
3. Bending moments are calculated with wind load and dead load using equation

$$M = \frac{wl^2}{8}$$

Where, M = moment in kN-m

W = load due to wind/dead load

L = span of the member

4. Section modulus in x and y direction are noted down, say z_{xx} and z_{yy}.
5. Maximum combined stresses are calculated using equation

$$\tau = \frac{M_w}{z_{xx}} + \frac{M_{dl}}{z_{yy}}$$

Where, Mw = Moment due to wind load

Mdl = Moment due to dead load

Maximum combined stresses should be less than permissible stresses.

3.2.3 Design of Monorail

1. Monorail was designed with two parts. One where it is simply supported by beams and the other where it is cantilevered.
2. Maximum bending moments are calculated for both the cases.

   Simply Supported:

   Moment due to self-weight $M_{dl} = wl^2/8$

   Moment due to point load of crane $M_l = Pl/4$

   Total Moment $M_{ss} = M_{dl} + M_l$

   Cantilever:

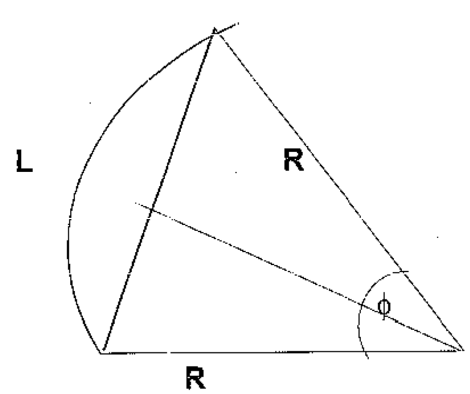
   Moment due to self-weight $M_{dl} = wl^2/2$

   Moment due to point load of crane $M_l = Pl$

   Total Moment $M_c = M_{dl} + M_l$

3. Final maximum bending moment applied $M = M_{ss} + M_c$
4. A trial I section is chosen and its design moment capacity is determined. Permissible stress bending stress σ_{cbc} can be adopted from IS 800-1984, table 6.1B.

   Design Moment $M_d = \sigma_{cbc} \times Z_{xx}$
   Where Z_{xx} = Section modulus in x direction
5. Ratio of design moment capacity with moment applied should be greater than 1.If not, we need to choose higher section.
6. For curves, the beam has to be checked for torsional moment using the following equations.



Torsional Moment M_τ = P.e (e = eccentricity)

Torsional stress $\tau = M_\tau/k$

Where k (Torsional constant) = $2bt_f^3 + d1t_w^3/L$

b = Flange width

t_f = thickness of flange

t_w = thickness of web

d1 = depth of web

Permissible torsion shear $\tau_{all} = 0.45f_y$

7. Check is done for combined bending moment and combined bending shear.

$M_d/M + \tau/\tau_{all} < 1$

3.2.4 Design of Steel Compression Members

ELASTIC BUCKLING OF EULER COLUMN

Assumptions:

- Material of strut - homogenous and linearly elastic
- No imperfections (perfectly straight)
- No eccentricity of loading
- No residual stresses

The governing differential equation is

$$\frac{d^2y}{dx^2} + \frac{P_{cr}}{EI}.y = 0$$

Lowest value of the critical load

$$P_{cr} = \frac{\pi^2 EI}{\ell^2}$$

$$\sigma_{cr} = \frac{P_{cr}}{A} = \frac{\pi^2 E\, I}{A\ell^2}$$

$$\sigma_{cr} = \frac{\pi^2 E\, r^2}{\ell^2} = \frac{\pi^2 E}{(\ell / r)^2} = \frac{\pi^2 E}{\lambda^2}$$

Where, A= area of cross-section, and

r = radius of gyration about the bending axis

λ = slenderness ratio

Strength of an Axially Loaded Compression
Members Maximum axial compression load permitted on a compression member

Where, P = axial compressive load (N),

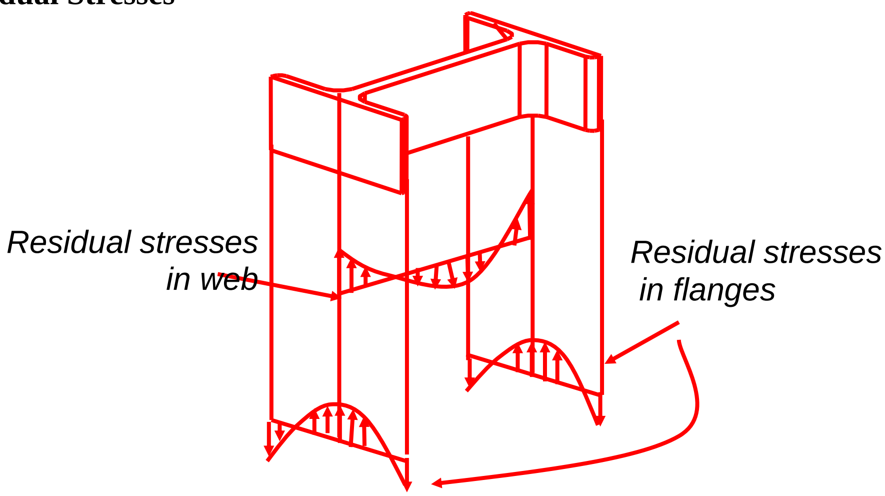
σ_{ac} = permissible stress in axial compression (MPa)

A = effective cross-sectional area of the member (mm^2)

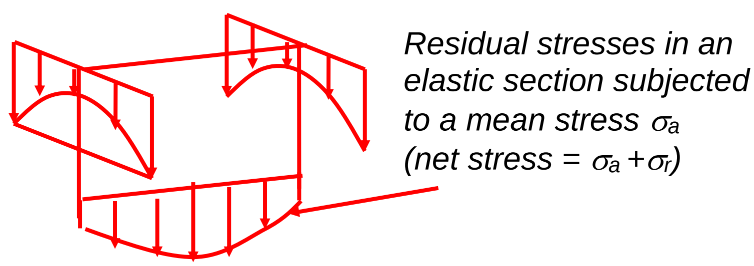
FACTORS AFFECTING STRENGTH OF A COLUMN IN PRACTICE:

- Effect of initial out of straightness
- Effect of eccentricity of applied loading
- Effect of residual stress
- Effect of a strain hardening and the absence of clearly defined yield point
- Effect of all features taken together

Residual Stresses



Residual stresses distribution (no applied load)



The influence of residual stresses

Fig 3.1. Residual stress distribution

DESIGN STRENGTH

The design compressive strength of a member is given by

$$P_d = A_e f_{cd}$$

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} \quad = \quad \chi f_y / \gamma_{m0} \quad \le f_y / \gamma_{m0}$$

$\phi = 0.5[1+\alpha (\lambda - 0.2)+ \lambda^2]$

f_{cd} = the design compressive stress,

λ = non-dimensional effective slenderness ratio,

$$= \sqrt{f_y \left(KL/r\right)^2 / \pi^2 E}$$

f_{cc} = Euler buckling stress = $\pi^2E/(KL/r)^2$

α = imperfection factor as in Table 7(IS 800:2007)

χ = stress reduction factor as in Table 8 (IS 800:2007)

Table Buckling Class of Cross-sections

Cross Section	**Limits**	**Buckling about axis**	**Buckling Curve**
Rolled I-Sections	***h/b*** **> 1.2 :** t_f **40 mm**	***z-z***	**a**
		y-y	**b**
	40 < t_f **<100**	***z-z***	**b**
		y-y	**c**
Welded I-Section	$t_f \leq$ **40 mm**	***z-z***	**b**
		y-y	**c**
	t_f **>40 mm**	***z-z***	**c**
		y-y	**d**
Hollow Section	**Hot rolled**	**Any**	**a**
	Cold formed	**Any**	**b**

Welded Box Section, built-up	**Generally**	**Any** **Any**	**b** **c**
Channel, Angle, T and Solid Sections		**Any**	**c**

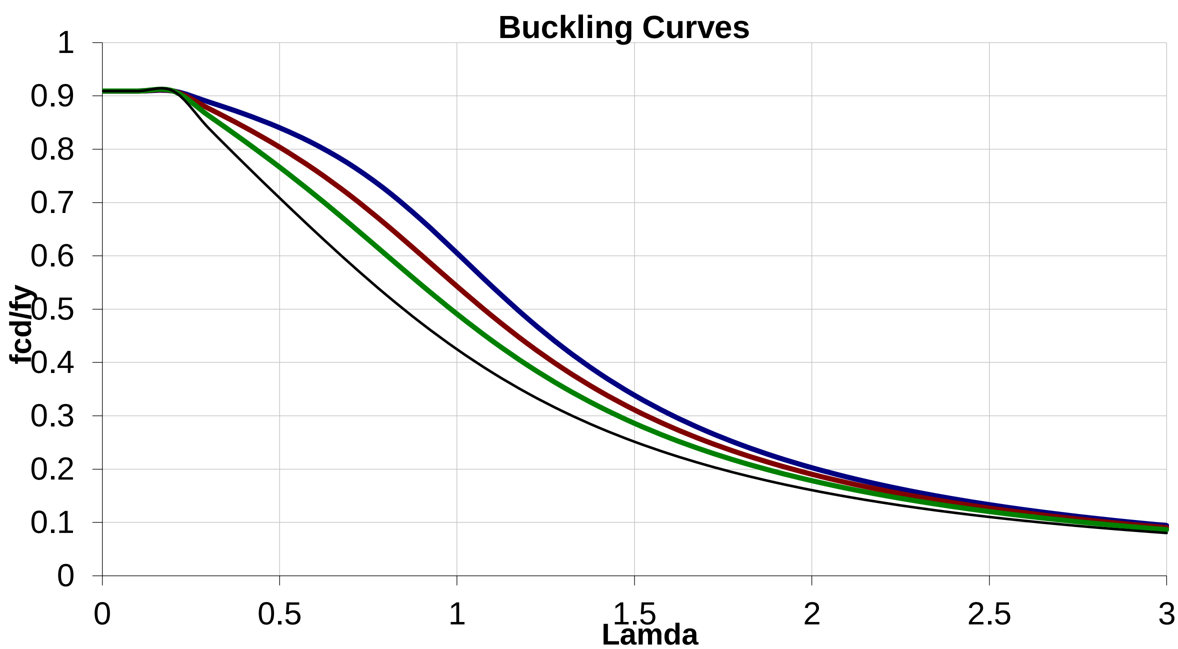


TABLE 7.1 IMPERFECTION FACTOR, *α*

Buckling Class	a	b	c	d
α	0.21	0.34	0.49	0.76

3.2.5 Design Of Cantilever Retaining Wall

In order to calculate the pressure exerted at any point on the wall, the following must be taken in account:

- Height of water table
- Nature & type of soil
- Subsoil water movements
- Type of wall
- Material used in the construction of wall

Analysis for dry back fills

Maximum pressure at any height, $p=k_a h$

Total pressure at any height from top, $P=1/2[k_a h]h = [k_a h^2]/2$

Bending moment at any height = $M=Pxh/3= [k_a h^3]/6$

Total pressure at bottom, $P_a= [k_a H^2]/2$

Total Bending moment at bottom, $M = [k_a H^3]/6$

Where, k_a= Coefficient of active earth pressure= $(1-\sin Ø)/(1+\sin Ø)=\tan^2 Ø$

$= 1/k_p$, coefficient of passive earth

Ø= Angle of internal friction or angle of repose

u=Unit weigh or density of backfill

If Ø = 30, k_a=1/3 and k_p=3. Thus k_a is 9 times k_p

Stability requirements of RW:

Following conditions must be satisfied for the wall

1. It should not overturn.
2. It should not slide.
3. It should not subside i.e. maximum pressure should not exceed the safe bearing capacity of the soil under working conditions.

Check against overturning

Factor of safety against overturning = MR / MO 1.55 (=1.4/0.9) Where, MR =Stabilising moment or restoring moment

MO =overturning moment

As per IS:456-2000,

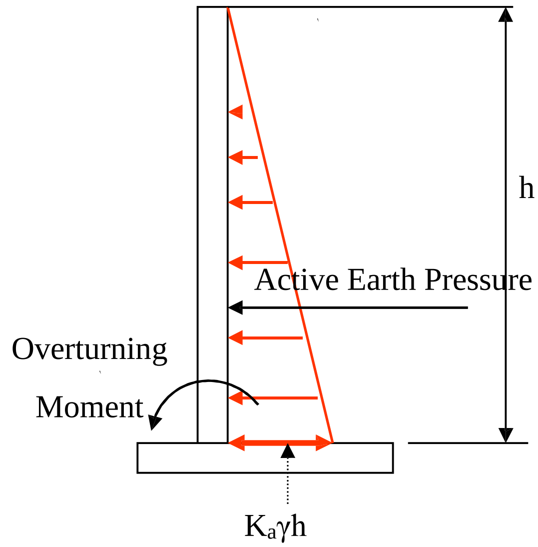


Fig 3.2. Various force on retaining wall

MR>1.2 MO, ch DL + 1.4 MO, ch IL

0.9 MR +1.4 MO, ch IL

Check against Sliding

FOS = Resisting force to sliding/Horizontal force causing sliding

= W/Pa x 1.55 (=1.4/0.9)

As per IS:456:2000

1.4 = (0.9□W)/Pa

Design of Shear key:

If the wall is not safe against sliding, then a shear key is to be provided. It is provided either below the stem or at the end of heel. It should not be provided at the end of toe. If shear key is provided, then it should be designed taking the effect of passive pressure.

In case the wall is unsafe against sliding pp= p tan2 (45 +1/2) = p kp

Where pp= Unit passive pressure on soil above shearing plane AB If

W= Total vertical force acting at the key base

Ø = shearing angle of passive resistance

R= Total passive force = pp x a

P_A=Active horizontal pressure at key base for H+a

W=Total frictional force under flat base

For equilibrium, R + W =FOS x PA

FOS= (R + W)/ PA x 1.55

Pressure below the wall

Consider the retaining wall as shown. All forces acting on the wall are shown. The moment of all forces at the end of toe is considered and the requirements of stability are to be established. For stability earth pressure at the end of the heel for the entire height of wall should be considered. The maximum and minimum pressure below the wall can be determined from the principles of static.

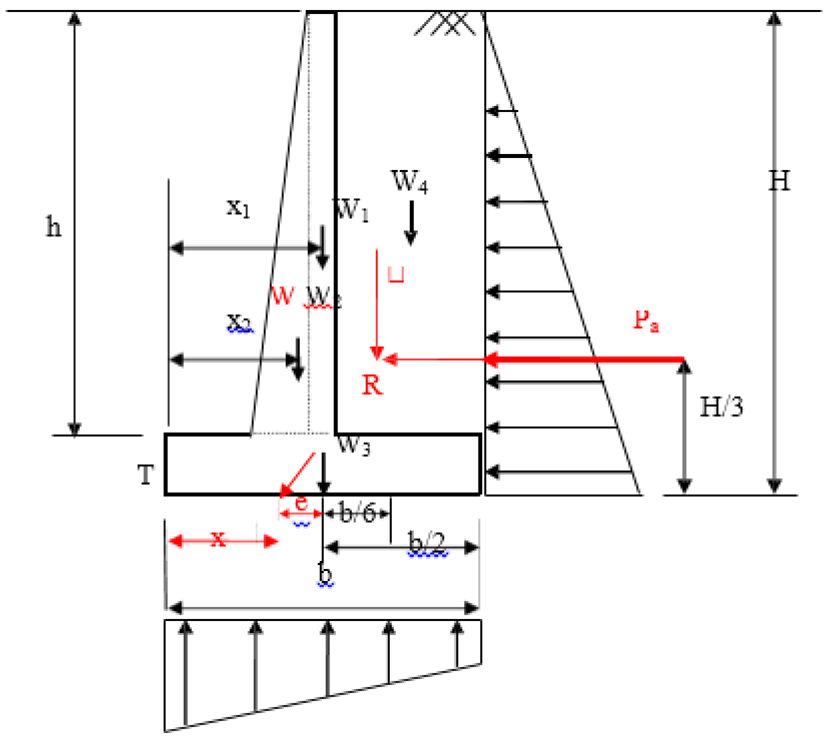


Fig 3.3. Various force on retaining wall

Preliminary Proportioning (T shaped wall)

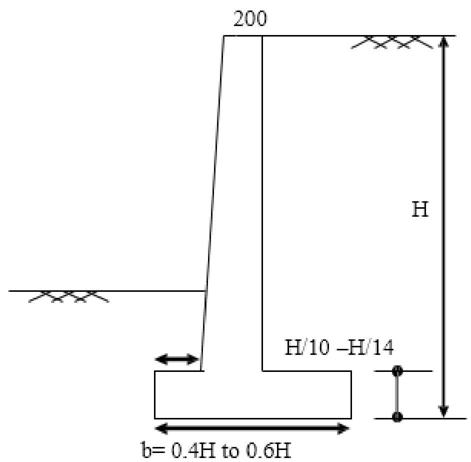
Following guidelines are to be followed for initial proportioning of wall with out surcharge. For surcharge and other cases, good text books should be followed.

Stem: Top width 200 mm to 400 mm

Base slab width b= 0.4H to 0.6H, and 0.6H to 0.75H for surcharged wall

Base slab thickness= H/10 to H/14

Toe projection= (1/3-1/4) Base width



Behaviour or structural action and design

All the three elements namely stem, toe and heel acts as cantilever slabs and hence the design and detailing principles are same as that of conventional cantilever slabs.

1. Stem design: Mu=partial safety factor x (ka H3/6)

2. Determine the depth d from Mu = Mu, lim=Qbd2

3. Determine the steel based on balanced or under reinforced design. Provide enough development length at the junction for all bars.

Curtailment of steel

Maximum steel is needed at the base where the BM is maximum. As the BM decreases towards the top, steel can be suitably curtailed at one or two levels. Usually steel is curtailed at one level where the steel quantity is about 50% or 67% of the base steel.

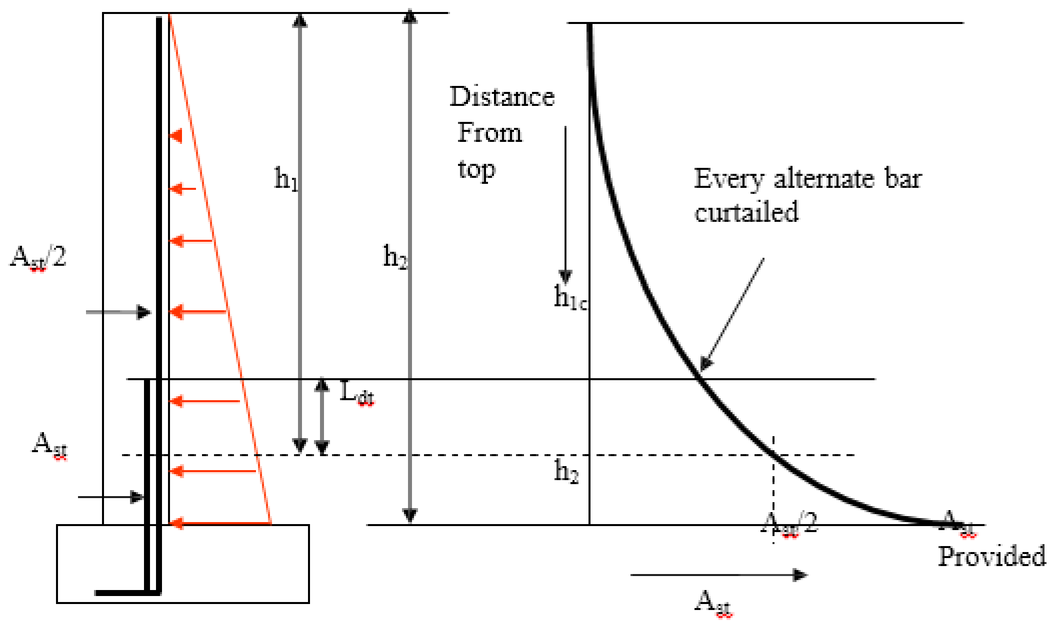


Fig 3.4. Curtailment of bars in retaining wall

3.2.6 Estimation of Cost

Ultimate goal of cost estimation is to determine the total quantity of the material used in the construction. Rate is provided in either in Rs/Kg or Rs/m^3, hence we have to find the total weight or total volume accordingly of materials used.

Basically, there are two parts in estimation

I. Estimation steel quantity

V = L X A

Where, V = Volume of steel

L = Length of steel section

A = Area of steel section

Total weight W = V x d

Where d = density of steel = 7850 kg/m^3

II. Estimation of retaining wall

Estimation of concrete quantity

Total Volume, $V = V_1 + V_2$

Where, V_1 = Volume of base slab

V_2 = Volume of vertical stem

Cost of concrete = R x V

Where, R = Rate of Concrete per cubic meter

Estimation of reinforcement

V = L X A

Where, V = Volume of steel

L = Length of steel section

A = Area of steel section

Total weight W = V x d

Where d = density of steel = =7850 kg/m^3

Estimation of Earthwork

V = A x L

Cost of excavation = V x R_1

Where, R = Rate of excavation per cubic meter

Cost of backfilling = V x R_2

Where, R = Rate of backfilling per cubic meter

4. LOADS AND FORCES

4.1 Dead Load

All permanent constructions of the structure form the dead loads. The dead load comprises of the weights of walls, partitions floor finishes, false ceilings, false floors and the other permanent constructions in the buildings. The dead load loads may be calculated from the dimensions of various members and their unit weights. the unit weights of plain concrete and reinforced concrete made with sand and gravel or crushed natural stone aggregate may be taken as 24 kN/m" and 25 kN/m" respectively. The dead load of steel sections are obtained according to SP-6.

4.2 Imposed Loads

Imposed load is produced by the intended use or occupancy of a building including the weight of movable partitions, distributed and concentrated loads, load due to impact and vibration and dust loads. Imposed loads do not include loads due to wind, seismic activity, snow, and loads imposed due to temperature changes to which the structure will be subjected to, creep and shrinkage of the structure, the differential settlements to which the structure may undergo.

4.3 Wind Load

Wind is air in motion relative to the surface of the earth. The primary cause of wind is traced to earth's rotation and differences in terrestrial radiation. The radiation effects are primarily responsible for convection either upwards or downwards. The wind generally blows horizontal to the ground at high wind speeds. Since vertical components of atmospheric motion are relatively small, the term 'wind' denotes almost exclusively the horizontal wind, vertical winds are always identified as such. The wind speeds are assessed with the aid of anemometers or anemographs which are installed at meteorological observatories at heights generally varying from 10 to 30 metres above ground.

4.3.1 Design Wind Speed (V)

The basic wind speed (V) for any site shall be obtained from and shall be modified to include the following effects to get design wind velocity at any height (V,) for the chosen structure:
a) Risk level;
b) Terrain roughness, height and size of structure; and
c) Local topography.
It can be mathematically expressed as follows:
Where:

$$\mathbf{V = Vb * k_1 * k_2 * k_3}$$

V_b = design wind speed at any height z in m/s;
k_l = probability factor (risk coefficient)
k = terrain, height and structure size factor and
k_3 = topography factor

Risk Coefficient (k_I Factor) gives basic wind speeds for terrain Category 2 as applicable at10 m above ground level based on 50 years mean return period. In the design of all buildings and structures, a regional basic wind speed having a mean return period of 50 years shall be used.

Terrain, Height and Structure Size Factor (k, Factor)
Terrain - Selection of terrain categories shall be made with due regard to the effect ofobstructions which constitute the ground surface roughness. The terrain category used in the design of a structure may vary depending on the direction of wind under consideration.Wherever sufficient meteorological information is available about the nature of wind direction, the orientation of any building or structure may be suitably planned.

Topography (k_s Factor) - The basic wind speed Vb takes account of the general level of site above sea level. This does not allow for local topographic features such as hills, valleys, cliffs, escarpments, or ridges which can significantly affect wind speed in their vicinity. The effect of topography is to accelerate wind near the summits of hills or crests of cliffs, escarpments or ridges and decelerate the wind in valleys or near the foot of cliff, steep escarpments, or ridges.

4.4 Calculations

Dead Load

a)	RCC Floor loads (200 thick slab) 0.2x 2500+100 (Including floor finish + screed)	600 kg/m²
b)	Sheeting weight	12 kg/m²
c)	RCC Roof loads (150 thick slab) 0.15x2500	375 kg/m²

Live Load

a)	RCC Floor	500 kg/m²
b)	RCC Roof (non-accessible)	255 kg/m²

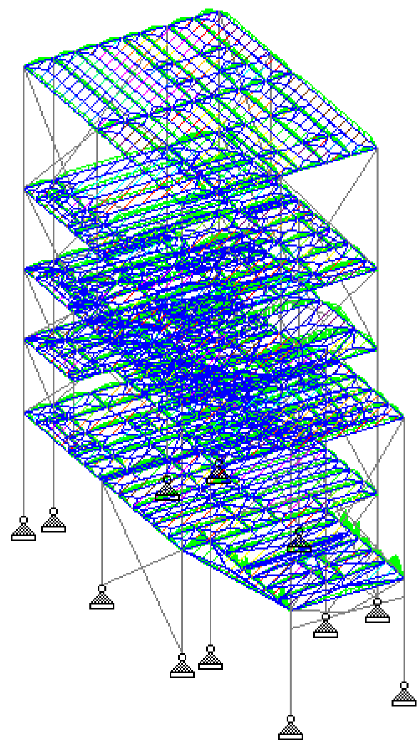


Fig 4.1 Application of dead load in STAADPro analysis

Wind Load Calculations

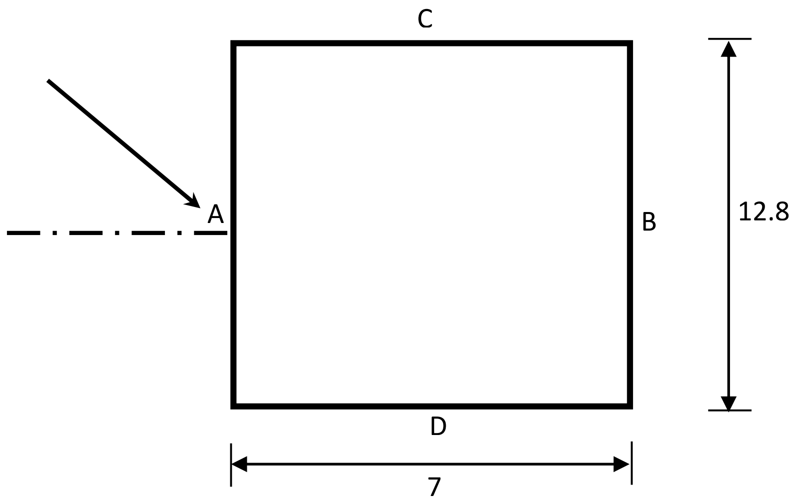
a)	Basic Wind speed (at 10.0m above MGL)	47 m/s
b)	Probability / Risk Factor (k1)	1.07
c)	Terrain Category	Category-II
d)	Topography Factor (k3)	1
Wind load calculations for Category-II & Class-A(size < 20m)		
e)	Terrain factor (k2)	1.06
	Design wind speed (Vz)= (Vbxk1xk2xk3)	53.31m/s
	Design wind Pressure (pz) = 0.6Vz2	171 kg/m²
f)	Length of building (l)	12.8 m
g)	Width of Building (w)	7 m
h)	Height of Building (h)	17.45 m

h/w = 17.45/7 = 2.49

Therefore, $3/2 < h/w < 6$

l/w = 12.8/7 = 1.83

Therefore, 3/2<l/w<4



Cladded Portion

<u>External Pressure Coefficients</u>

C_{pe} Value	A	B	C	D
For Θ = 0	0.7	-0.4	-0.7	-0.7
For Θ = 90	-0.5	-0.5	0.8	-0.1

<u>Internal Pressure Coefficient</u>

	(pressure)		(suction)
C_{pi} value :	0.5	or	-0.5

Wind Pressure p = 1524.1 N/ m^2

Wind Force = p*($C_{pe} - C_{pi}$)

For Θ = 0 & +ve C_{pi}

Faces	$C_{pe} - C_{pi}$	Force (N/ m²)
A	0.2	304.82
B	-0.9	-1371.69
C	-1.2	-1828.92
D	-1.2	-1828.92

For Θ = 0 & -ve C_{pi}

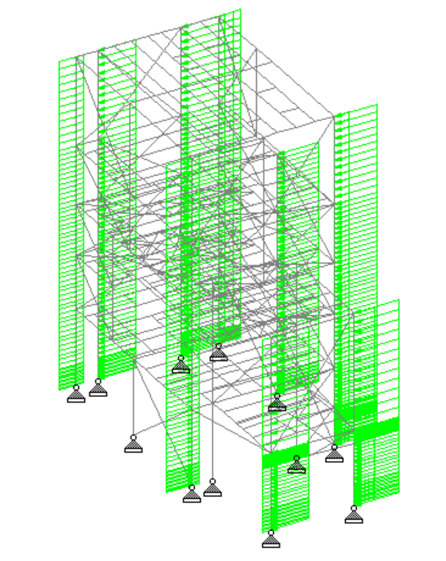
Faces	$C_{pe} - C_{pi}$	Force (N/ m²)
A	1.2	1828.92
B	0.1	152.41
C	-0.2	-304.82
D	-0.2	-304.82

For Θ = 90 & +ve C_{pi}

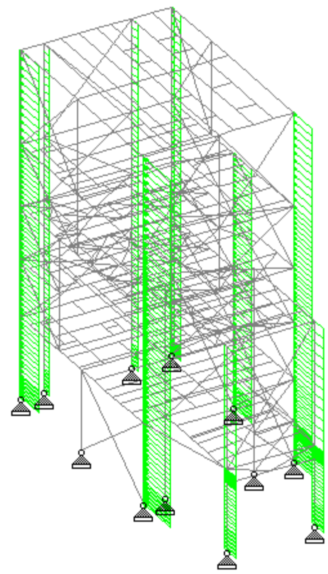
Faces	$C_{pe} - C_{pi}$	Force (N/ m²)
A	-1	-1524.1
B	-1	-1524.1
C	0.3	457.23
D	-0.6	-914.46

For Θ = 90 & -ve C_{pi}

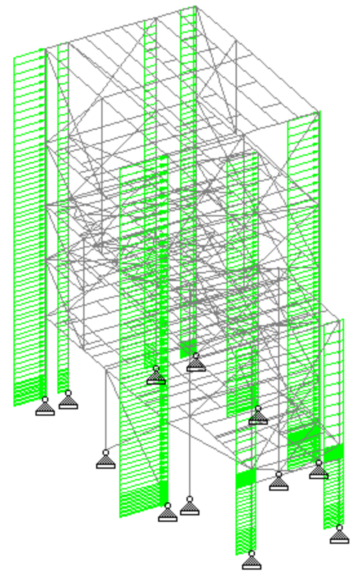
Faces	$C_{pe} - C_{pi}$	Force (N/ m²)
A	0	0
B	0	0
C	1.3	1981.33
D	0.4	609.64



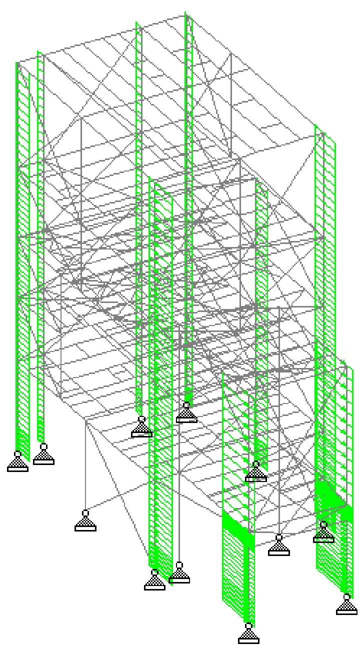
(a) Θ = 0 & +ve C_{pi}



(b) Θ = 90 & +ve C_{pi}



(c) Θ = 0 & -ve C_{pi}



(d) Θ = 90 & -ve C_{pi}

Fig 4.2 Application of wind load in STAADPro analysis

5. STAADPro ANALYSIS AND RESULT

5.1 Project Design Approach

The two dimensional drawing was rendered to get the three dimensional drawing. Beams and purlins of roof truss were added. Plates were assigned to the roof, first storey floor and second storey floor.

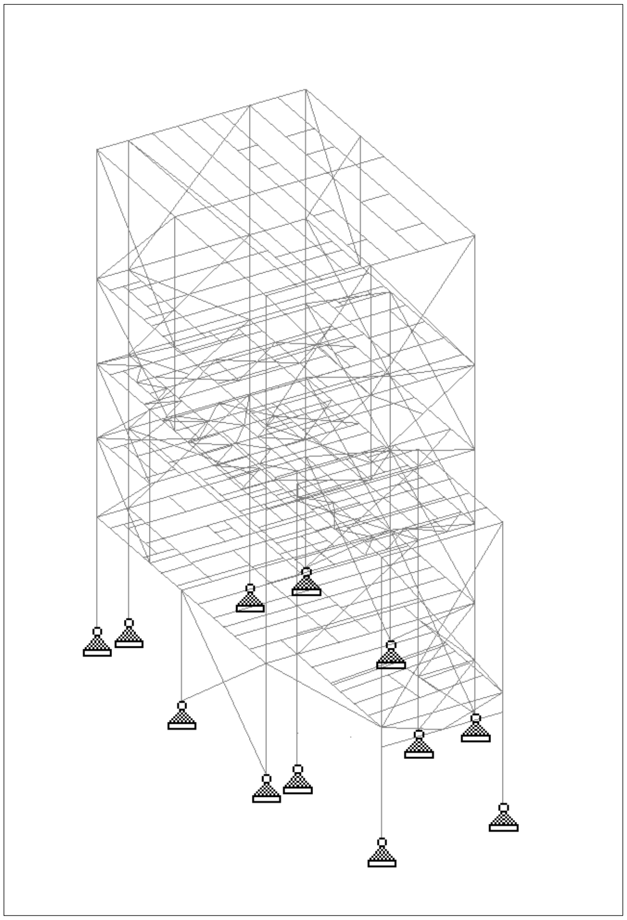


Fig 5.1 STAADPro model of TP8 structure

After analyzing on the structure TP8 & modeling TP8- A on STAADPRO, results were printed out showing various beams and columns either passing the test or failing. Following results were used to modify the existing as well as newly created structure.

Below figure shows the modeling of the newly created structure attached with the existing structure.

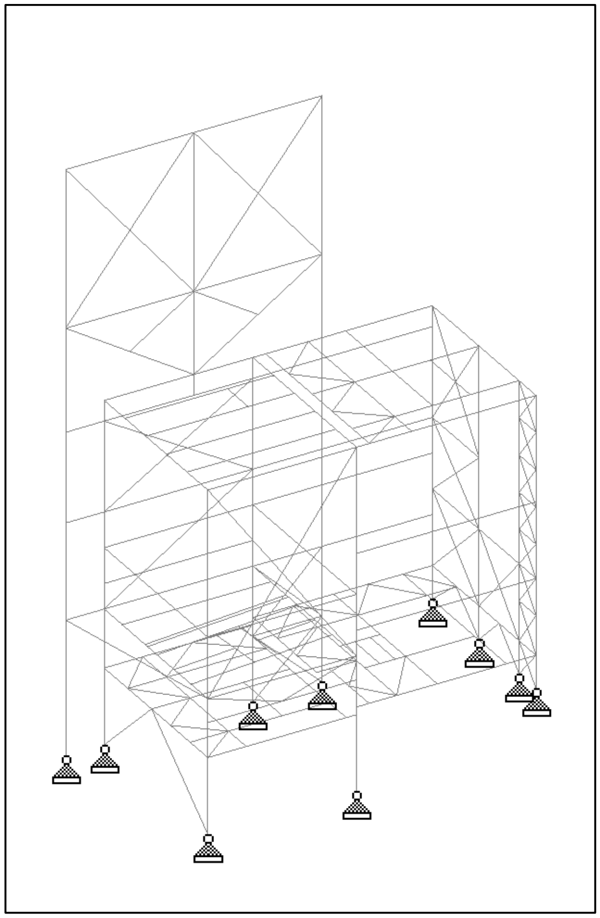


Fig 5.2 STAADPro model of TP8-A structure

5.2 STAADPro Analysis

Before analyzing the TP8-A structure, its columns and beams were assigned an abstract member sections like ISMB450 for beams, ISMB600 for the columns and ISA 100x100x8 for truss members. The analysis result shows whether they are appropriate for the structure or has to be replaced with a larger member section.

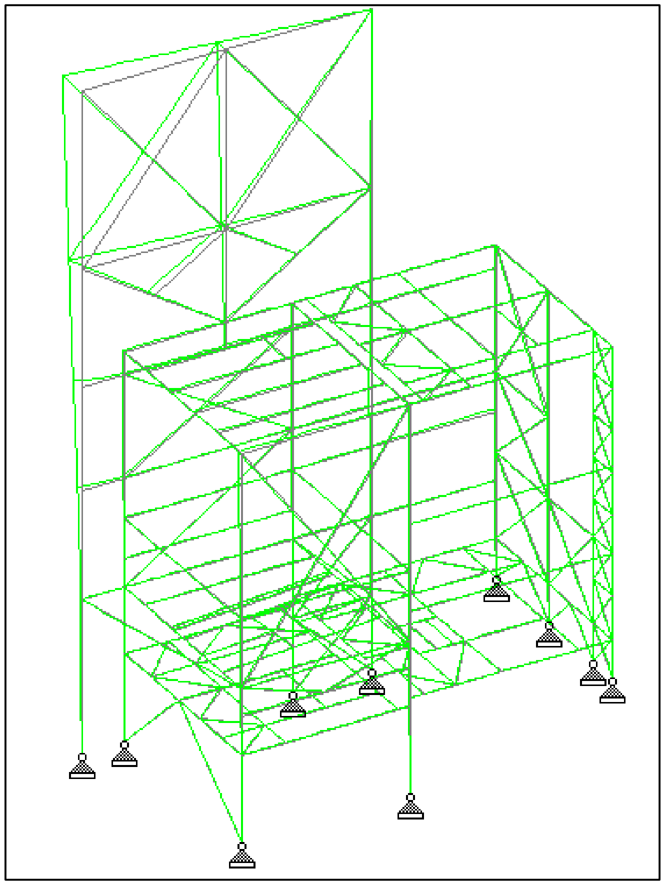


Fig 5.3 Deflection shown in STAADPro

As seen in the above image, the deflection is very less especially where braces have been provided. That fulfills the requirement of braces which is minimizing lateral deflection.

Optimization of the design in STAADPRO was done so that members which were failing would be replaced by the optimized members in the designed model. Most of the members failing were the abstract sections we have assigned. They were automatically replaced by an optimized member section by the software. Following figures illustrate the replaced sections.

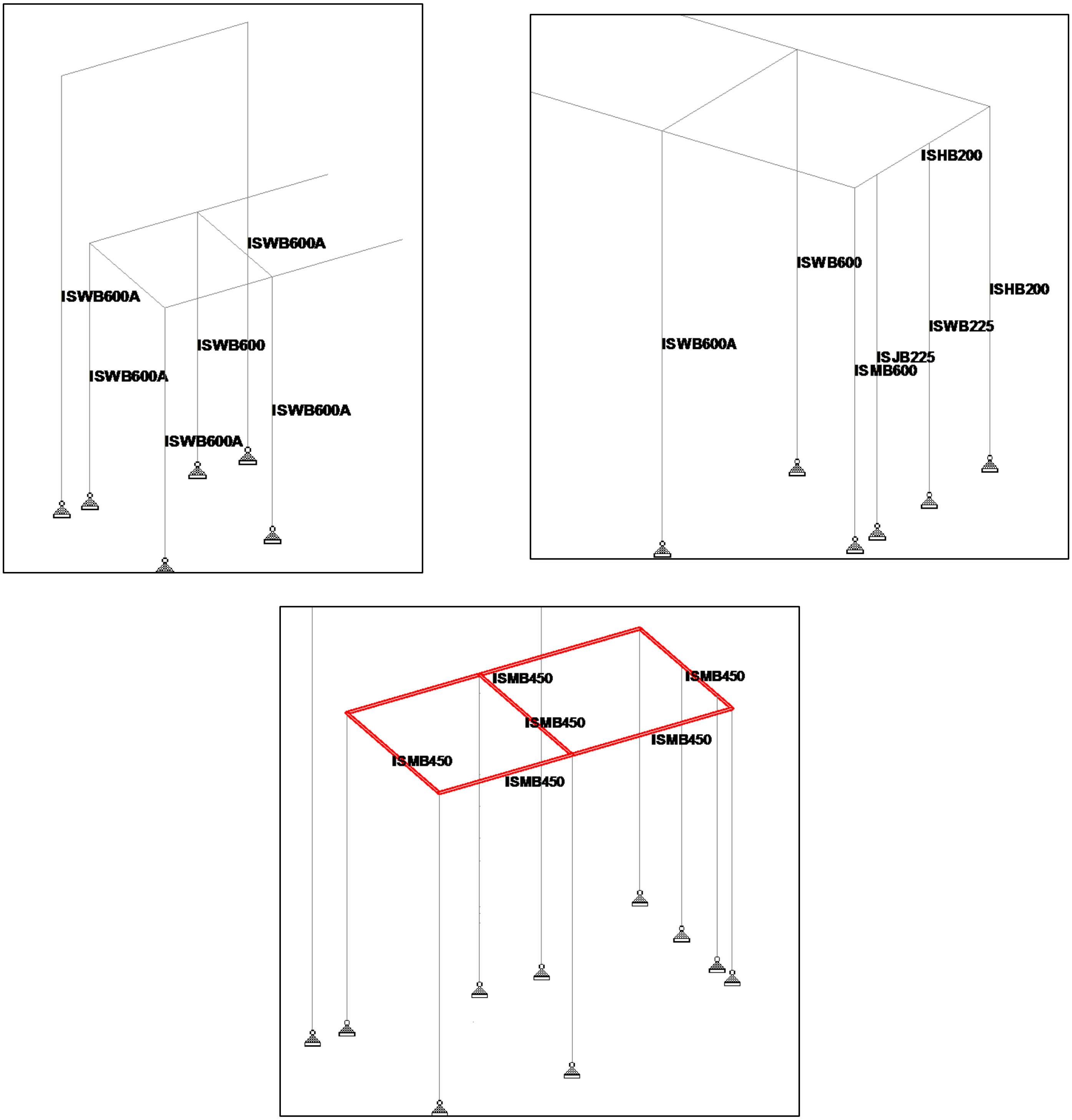


Fig 5.4 Designed section for TP8-A structure

Above designs were passed by the analysis done in STAADPRO. The data can be used in construction of the structure TP8-A**.** All the sections for each beam number is provided appendix C.

6. Manual Design and Calculations

6.1 Design of Base Plates

Maximum Compression P = 10 ton

Maximum Shear (Fx) = 10 ton

Maximum Shear (Fy) = 10 ton

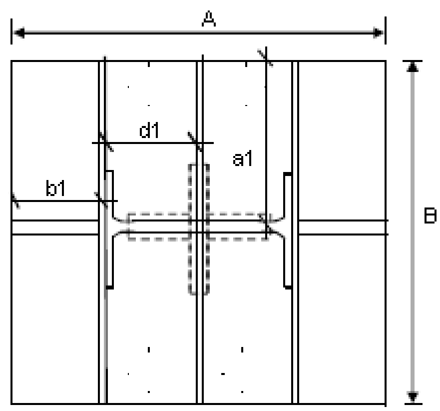


Fig 6.1 Schematic diagram of base plate

Designing for Section ISMB 600

Concrete Grade: M25

Assuming Size of base plate

600	**X**	**750**
B	**X**	**A**

Area of base plate = 4500 cm^2

Bearing Pressure developed q = P/Area = 10/4500

= 2.22 kg/cm^2

Allowable bearing pressure = 62.5 kg/cm^2

Check for Bending of Base Plate

a1	300 mm
d1	470 mm
ASPECT RATIO	0.6
α_3	0.06
MOMENT IN ZONE II (α_3 X q X d12)	294.53 kg-cm
LENGTH IN CANTILEVER PORTION, L	75 mm
MOMENT IN CANTILEVER PORTION (W x L x L/2)	62.5 kg-cm
HENCE, MAX MOMENT	294.53 kg-cm
Let the thickness of base plate required be t = $\sqrt{(6M/R)}$	1.03 cm
THICKNESS ADOPTING	15 mm

Design of Bolts

MAXIMUM TENSION ON PLATE	2 ton
NUMBER OF BOLTS	4
TENSION PER BOLT	0.5 ton
ADOPTING DIAMETER	20 mm
Check for shear	
MAXIMUM SHEAR IN X AXIS(FROM STAAD)	10.00 ton
MAXIMUM SHEAR IN Z AXIS(FROM STAAD)	10.00 ton
SO, RESULTANT SHEAR FORCE	14.1 ton
SO, SHEAR PER BOLT	3.53 ton
AREA OF EACH BOLT	3.14 cm^2
SO, SHEAR CAPACITY OF EACH BOLT	2.51 ton

HENCE, SAFE

Designing for Section ISHB 200

Concrete Grade: M25

Assuming Size of base plate

250	**X**	**300**
B	**X**	**A**

Area of base plate = 750 cm^2

Bearing Pressure developed q = P/Area = 10/750

= 133.33 kg/cm^2

Allowable bearing pressure = 62.5 kg/cm^2

Check for Bending of Base Plate

a1	120 mm
d1	220 mm
ASPECT RATIO	0.5
α_3	0.06
MOMENT IN ZONE II (α_3 X q X d12)	3872 kg-cm
LENGTH IN CANTILEVER PORTION, L	50 mm
MOMENT IN CANTILEVER PORTION (W x L x L/2)	1666.67 kg-cm
HENCE, MAX MOMENT	3872 kg-cm
Let the thickness of base plate required be t = $\sqrt{(6M/R)}$	3.8 cm
THICKNESS ADOPTING	40 mm

Design of Bolts

MAXIMUM TENSION ON PLATE	2 ton
NUMBER OF BOLTS	4
TENSION PER BOLT	0.5 ton
ADOPTING DIAMETER	20 mm
Check for shear	
MAXIMUM SHEAR IN X AXIS(FROM STAAD)	50 ton
MAXIMUM SHEAR IN Z AXIS(FROM STAAD)	50 ton
SO, RESULTANT SHEAR FORCE	70.7 ton
SO, SHEAR PER BOLT	17.67 ton
AREA OF EACH BOLT	3.14 cm^2
SO, SHEAR CAPACITY OF EACH BOLT	2.51 ton

HENCE, SAFE

Designing for Section ISWB 600 &ISWB 600A

Concrete Grade: M25

Assuming Size of base plate

600	X	750
B	**X**	**A**

Area of base plate = 4500 cm^2

Bearing Pressure developed q = P/Area = 10/4500

= 2.22 kg/cm^2

Allowable bearing pressure = 62.5 kg/cm^2

Check for Bending of Base Plate

a1	300 mm
d1	470 mm
ASPECT RATIO	0.6
α_3	0.06
MOMENT IN ZONE II (α_3 X q X d12)	294.53 kg-cm
LENGTH IN CANTILEVER PORTION, L	75 mm
MOMENT IN CANTILEVER PORTION (W x L x L/2)	62.5 kg-cm
HENCE, MAX MOMENT	294.53 kg-cm
Let the thickness of base plate required be t = √(6M/R)	1.03 cm
THICKNESS ADOPTING	15 mm

Design of Bolts

MAXIMUM TENSION ON PLATE	2 ton
NUMBER OF BOLTS	4
TENSION PER BOLT	0.5 ton
ADOPTING DIAMETER	20 mm
Check for shear	
MAXIMUM SHEAR IN X AXIS(FROM STAAD)	10.00 ton
MAXIMUM SHEAR IN Z AXIS(FROM STAAD)	10.00 ton
SO, RESULTANT SHEAR FORCE	14.1 ton
SO, SHEAR PER BOLT	3.53 ton
AREA OF EACH BOLT	3.14 cm^2
SO, SHEAR CAPACITY OF EACH BOLT	2.51 ton

HENCE, SAFE

Designing for Section ISWB 225 & ISJB 225

Concrete Grade: M25

Assuming Size of base plate

250	**X**	**300**
B	**X**	**A**

Area of base plate = 750 cm^2

Bearing Pressure developed q = P/Area = 10/750

= 133.33 kg/cm^2

Allowable bearing pressure = 62.5 kg/cm^2

Check for Bending of Base Plate

a1	120 mm
d1	220 mm
ASPECT RATIO	0.5
α_3	0.06
MOMENT IN ZONE II (α_3 X q X d12)	3872 kg-cm
LENGTH IN CANTILEVER PORTION, L	50 mm
MOMENT IN CANTILEVER PORTION (W x L x L/2)	1666.67 kg-cm
HENCE, MAX MOMENT	3872 kg-cm
Let the thickness of base plate required be t = √(6M/R)	3.8 cm
THICKNESS ADOPTING	40 mm

Design of Bolts

MAXIMUM TENSION ON PLATE	2 ton
NUMBER OF BOLTS	4
TENSION PER BOLT	0.5 ton
ADOPTING DIAMETER	20 mm
Check for shear	
MAXIMUM SHEAR IN X AXIS(FROM STAAD)	50 ton
MAXIMUM SHEAR IN Z AXIS(FROM STAAD)	50 ton
SO, RESULTANT SHEAR FORCE	70.7 ton
SO, SHEAR PER BOLT	17.67 ton
AREA OF EACH BOLT	3.14 cm^2
SO, SHEAR CAPACITY OF EACH BOLT	2.51 ton

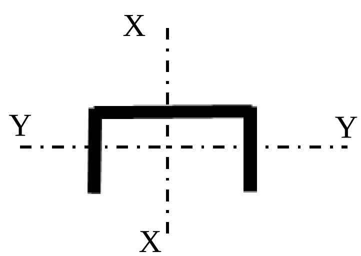
HENCE, SAFE

SUMMARY

Column Section	Base Plate (mm)			Bolt Diameter (mm)
	L	B	t	
ISMB 600	600	750	20	20
ISHB 200	250	300	40	20
ISWB 600	600	750	20	20
ISWB 600A	600	750	20	20
ISWB 225	250	300	40	20
ISJB 225	250	300	40	20

6.2 Design of Cladding

Assuming rail section: MC 150



MAXIMUM SPAN	Lx	7 m
	Ly	3.5 m
MAXIMUM SPACING		1.4 m
SELF WEIGHT OF RAIL SECTION (MC 150)		16.4 kg/m
SELF WEIGHT OF SHEETING		9 kg/m^2
I.E LOAD ON RAIL 1.4X9=		12.6 kg/m
TOTAL DEAD LOAD		29 kg/m
WIND LOAD ON RUNNERS		
MAX. WIND PRESSURE COEFFICIENT		1
WIND PRESSURE		1 kg/m^2
WIND LOAD 1x1x1.4		1.4 kg/m
BENDING MOMENT ABOUT X-X AXIS 1.4 x 7 x 7/8		8.58 kg-m
BENDING MOMENT ABOUT X-X AXIS 29 x 3.5 x 3.5/8		44.41 kg-m
SELECTING SECTION MC 150 (WITH SAG ROD)	Zxx	105 cm^3
	Zyy	19.5 cm^3
MAX COMBINED STRESS 8.58X100/105+44.41X100/19.5=		235.92 kg/cm^2

Permissible Combined stress = 2194.5 kg/cm^2

HENCE, SAFE

6.3 Design of Monorail

CASE 1: Simply Supported

MAX. SPAN, L	3 m
CAPACITY OF MONORAIL, (ELECTRICAL HOIST)	2 ton
CAPACITYT OF MONORAIL REQUIRED, W	2.5 ton
SELF WEIGHT ASSUMED, W	50 kg/m
BENDING MOMENT DUE TO MONORAIL = WL/4	1.88 t-m
MOMENT DUE TO SELF WT. = $WL^2/8$	0.06 t-m
TOTAL B.M, M	1.94 t-m

CASE 2: Cantilevers

MAX. SPAN, L	3.25 m
SELF WEIGHT ASSUMED, W	50 kg/m
BENDING MOMENT DUE TO MONORAIL = WL	8.125 t-m
MOMENT DUE TO SELF WT. = $WL^2/2$	0.26 t-m
TOTAL B.M, M	8.38 t-m
HENCE MOMENT, M_D	8.38 t-m

Trying with section **ISMB 350**

SECTIONAL AREA	66.71 cm^2
DEPTH, D	350 mm
MEAN THICKNESS OF COMP FLANGE, T	14.2 mm
DEPTH OF WEB, D1	321.6 mm
WEB THICKNESS, T	8.1 mm
WIDTH OF FLANGE, B	140
T/t	1.75
d1/t	39.7
I_{xx}	13630.3 cm^4
Z_{xx}	778.9 cm^3
r_{yy}	2.84 cm

L_e	325 cm
L_e/r_{yy}	114.44
FROM IS 800-1984, TABLE 6.1B	
PERMISSIBLE BENDING STRESS, ΣBC,ALLOW	1167.2 kg/cm^2
HENCE DESIGN MOMENT, M_R	**9.09 t-m**
RATIO M_R /M_D	1.08
	SAFE
Check for Torsional Moment	
RADIUS OF CURVATURE OF MONORAIL	3 m
LENGTH OF BEAM ALONG CURVATURE	1.75 m
Ø	33.42 deg
ECCENTRICITY DUE TO CURVATURE	0.12 m
TORSIONAL MOMENT	0.3 t-m
TORSIONAL CONSTANT K (2/3X140X14.2^3+1/3X321.6X8.1^3)	32.42 cm^4
TORSIONAL SHEAR CALCULATED 30000/32.42X0.81=	749.54 kg/cm^2
PERMISSIBLE TORSIONAL SHEAR 0.45X250	1125kg/cm^2
HENCE COMBINED BENDING + TORSIONAL STRESS	0.88
	<1 **SAFE**

6.4 Design of Retaining wall

As per the design procedure mentioned above, following is the summary of various forces calculated for the design.

TYPE OF FORCES	FORCES	A_{ST} PROVIDED
ACTIVE PRESSURE	196.528 kN	
OVERTURNING MOMENT	441.967 kNm	
TOTAL VERTICAL FORCE	380.496 kN	

TOTAL MOMENT GENERATED BY VERTICAL FORCE	615.162 kNm	
STABILIZING MOMENT	1043.83 kNm	
SOIL PRESSURE AT FOOTING BASE:		
Q_{MAX}	159.11 kN/m^2	
Q_{MIN}	15.400kN/m^2	
DESIGN SHEAR FORCE IN TOE SLAB	186.98 kN/m	
DESIGN MOMENT FOR TOE SLAB	204.21 kNm/m	1021.86 mm^2
DESIGN SHEAR FORCE IN HEEL SLAB	219.50 kN/m	
DESIGN MOMENT FOR HEEL SLAB	306.62 kNm/m	1557.12 mm^2
MAXIMUM DESIGN MOMENT IN VERTICAL STEM	508.52 kNm/m	2386.8 mm^2

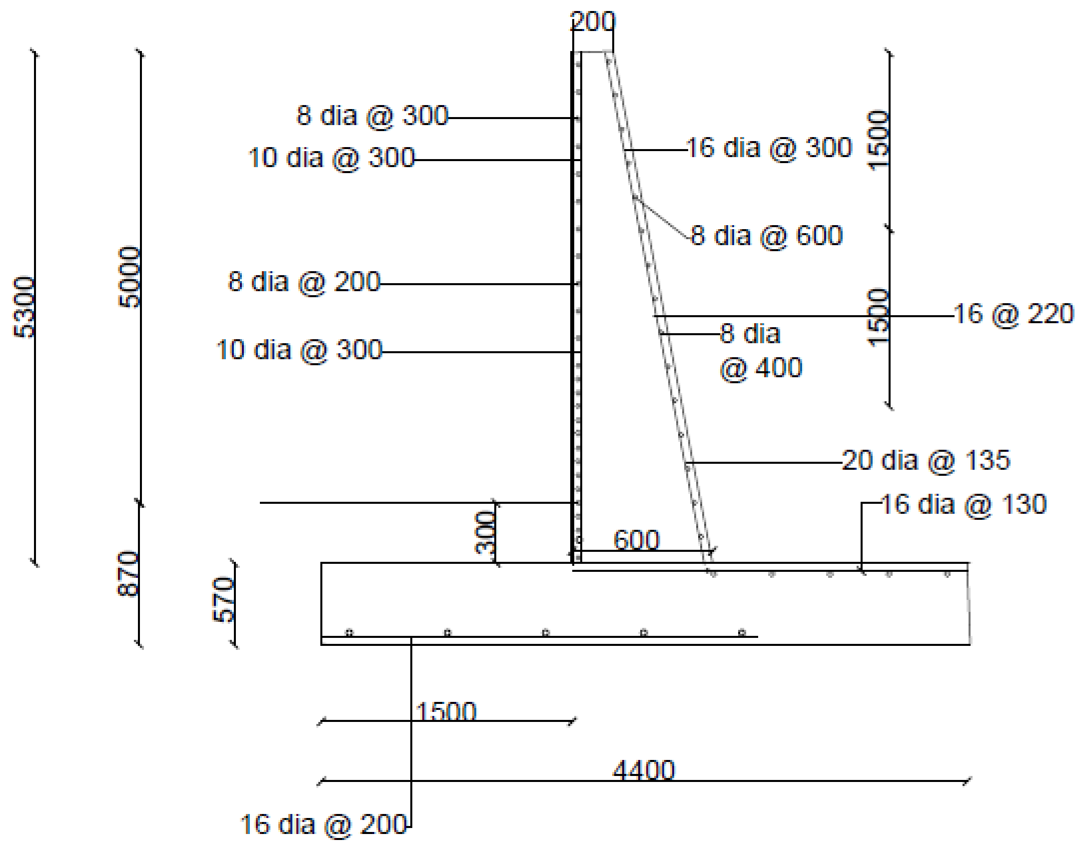


Fig 6.2 Detailing of retaining wall

6.5 Cost Estimation

I. Steel Section

Steel Sections					
Sections	Area (cm^2)	Weight per meter (kg)	Length (m)	Quantity	Total Weight (ton)
ISMB 600	156.21	122.6	1.55	18	3.42054
ISHB 200	50.94	40	1.5	24	1.44
ISWB 600	170.38	133.7	1.4	51	9.54618
ISJB 225	16.28	12.8	2.15	18	0.49536
ISMB 250	47.65	37.3	5.8	11	2.37974
ISMB 350	66.71	52.4	5.8	23	6.99016
ISHB 350	85.91	67.4	1.95	4	0.52572
ISLB 200	25.27	19.8	1.6	4	0.12672
ISLB 75	7.7	6.1	1.7	13	0.13481
ISMB 500	110.74	86.9	5	7	3.0415
ISHB 150	38.98	30.6	4.36	7	0.933912
ISHB 250	64.96	51	4.3	9	1.9737
ISLB 175	21.3	16.7	1.65	8	0.22044
ISWB 250	52.05	40.9	3.9	3	0.47853
ISA 130*130	29.82	23.4	2.9	6	0.40716
ISA 130*130	25.06	19.7	3.8	28	2.09608
ISA 100*100	15.39	12.1	1.6	28	0.54208
ISA 75*75	7.27	5.7	2	2	0.0228
ISA 70*70	6.77	5.3	1.87	3	0.029733
ISA 65*65	6.25	4.9	1.87	1	0.009163
ISA 55*55	5.27	4.1	1.7	13	0.09061
Total Weight					**34.90**

Base Plates					
Sections	Length (mm)	Breadth (mm)	Thickness (mm)	Volume (cm^3)	Total Weight (kg)
ISMB600	600	750	20	9000	0.7065
ISHB200	250	300	40	3000	0.2355
ISWB600	600	750	20	9000	0.7065
ISWB600A	600	750	20	9000	0.7065

ISWB225	250	300	40	3000	0.2355
ISJB225	250	300	40	3000	0.2355
Total Weight					**2.82**

Cost of Steel = Rs 42/kg

Cost of Steel Sections = 34.90*1000*42 = Rs 1466007.39

Cost of Base plates = 2.82*42 = Rs 118.69

Cost of Monorail (ISMB 350)

Area = 66.71 cm^2

Weight = 52.4 kg

Length = 3 + 3 + 1.75 = 7.75 m

Total Weight = 406.1 kg

Cost = 42 * 406.1 = Rs17056

Total cost of steel structure = Rs 14,83,182

II. Retaining Wall

Concrete Cost Estimation				
Element	Total Length (m)	Volume of concrete m^3	Rate per m^3	Total Cost
Base Slab				
Rectangle	21	52.668	3500	184338
Stem				
Rectangle	21	22.26	3500	77910
Triangle	21	22.26	3500	77910
Total				**262254**

Reinforcement Cost Estimation						
Element		Total Length (m)	Volume of steel m^3		Rate per m^3	Total Cost
Toe Slab	1021.86	21	0.021	172.74	42	7255.30
Heel Slab	1557.12	21	0.03	263.23	42	11055.70
Vertical Stem	2386.8	21	0.05	403.48	42	16946.51
Shear at base of Stem	507.6	21	0.01	85.80	42	3604.01
Temperature and Shrinkage reinforcement	480	21	0.01	81.14	42	3408.04
Toe Slab	1021.86	21	0.021	172.745433	42	7255.30
Total						**42269.59**

Cost of Earthwork

Volume of excavation = 54.38 m^3

Rate = 20 per m^3

Total Cost = Rs 10847.76

Total cost of Retaining wall = Rs 3,69,610.15

Total Cost of Structure = Rs 18,52,793

7. CONCLUSION

At the end, the TP8-A steel building was designed successfully according to the IS 800:2007. The section chosen for base plate, channel sections and columns are adequate to support the loading imposed on the building. The adequacy of the section has been checked and verified by STAADPro simulation and hand calculation.

In our Project, while designing the structure we observed that some of the columns and beams already existed need to be welded with plate girders in order to strengthen so that it can hold of the extra load it is taking from the new structure. Two Columns in front of the structure (face 1) needed extra thickness at their flange and web. Rest of the load can be transferred to the newly built columns by using high beams with larger web and flange thickness (like ISMB600, ISWB600A etc.).More detailed data of all beams and columns sections adopted is provided in appendix C.

While designing claddings, assuming the channel section **MC150** was coming out to be safe. This section can be used on all sides of the structure. Cladding can be installed on at equal distances with maximum length of 7m. Base plates were successfully designed for each and every column. Also, while designing the monorail, after doing trial with different I sections, **ISMB 350** was coming out to be safe and most economic for handling crane loads.

Cost estimation of project was also done successfully. Structure is quite economical to build. Cost of total steel structure including beams, columns and base is Rs 14,83,182. Cost of retaining wall including concrete work, reinforcement and excavation is Rs 3,69,160.Hence total cost comes out to be **Rs 18,52,793**.

While working on this project, we were able to:

1. Learn various techniques and tricks in order to analyze and design a steel structure on software STAADPRO.
2. We learn various design procedures and concepts of designing of steel structures mentioned in IS 800:2007.
3. Learn new concepts and methods regarding wind load action on a high rise building (H>10m) by referring IS875(part-3)
4. Also, we can now easily calculate wind load action on a steel structure.
5. Not to mention, working in a team was also an experience from which we learned working as a team and coordinating among teammates.

APPENDIX A

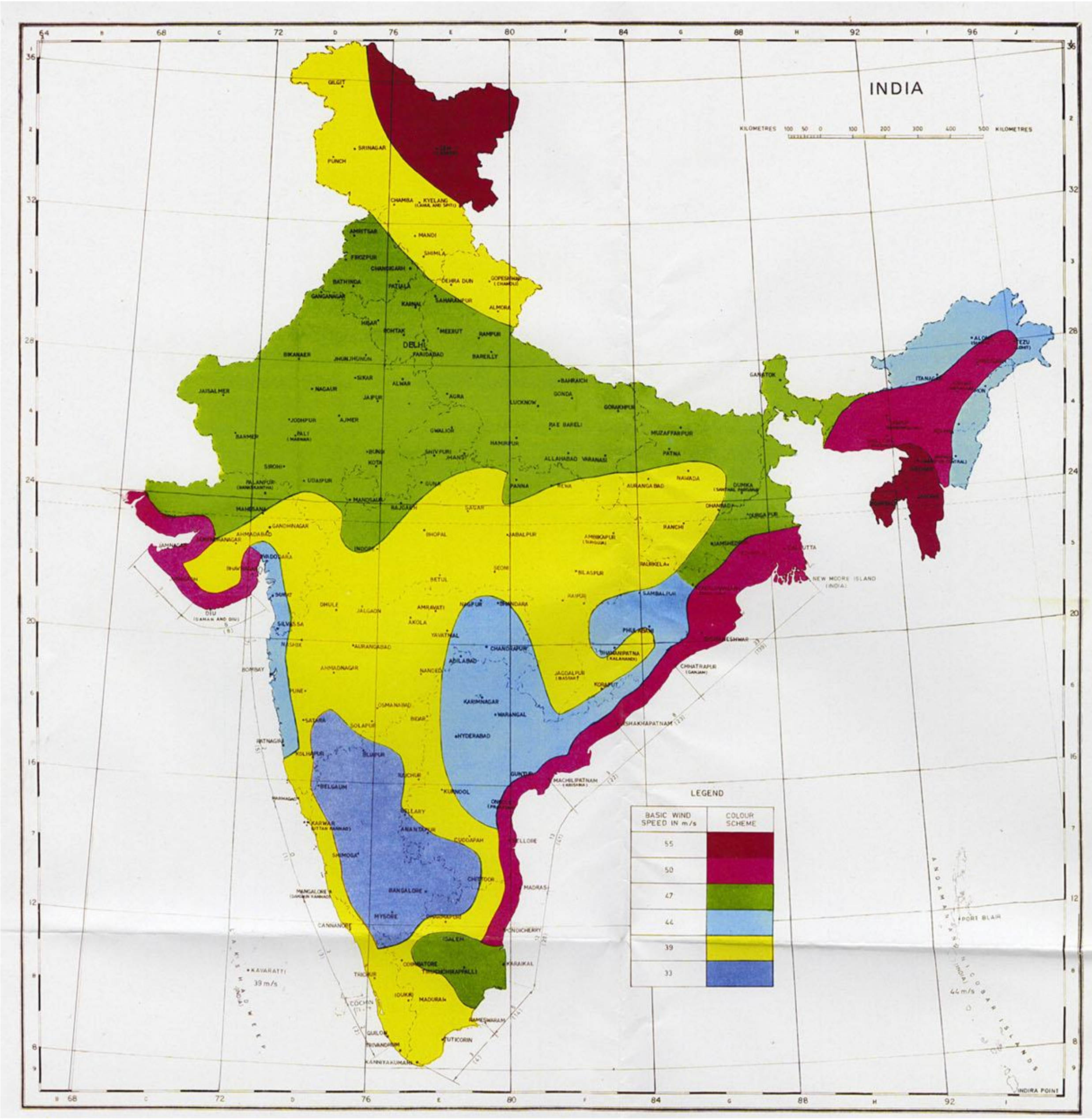
Table 1. Section Property Table

Destination	**Weight per Metre**	**Sectional Area**	**Depth of Section**	**Width of Flange**	**Thickness of Flange**	**Thickness of Web**	**Radii of Gyration**		**Section Modulus**	**Plastic modulus**	**Shape Factor**
			(D)	**(b_f)**	**(t_f)**	**(t_w)**	**(r_z)**	**(r_y)**	**(Z_{ez})**	**(Z_{pz})**	**(Z_{pz} / Z_{ez})**
	kg/m	cm^2	mm	mm	mm	cm	cm	cm	cm^3	cm^3	
ISWB600	145.1	184.86	600	250	23.6	11.8	25.01	5.35	3854.2	4341.63	1.1265
ISWB600	133.7	170.38	600	250	21.3	11.2	24.97	5.25	3540	3986.66	1.1262
ISMB600	122.6	156.21	600	210	20.8	12	24.24	4.12	3060.4	3510.63	1.1471
ISWB550	112.5	143.34	550	250	17.6	10.5	22.86	5.11	2723.9	3066.29	1.1257
ISMB500	103.7	132.11	550	190	19.3	11.2	22.16	3.73	2359.8	2711.98	1.1492
ISWB500	95.2	121.22	500	250	14.7	9.9	20.77	4.96	2091.6	2351.35	1.1242
ISMB500	86.9	110.74	500	180	17.2	10.2	20.21	3.52	1808.7	2074.67	1.1471
ISHB450	92.5	117.89	450	250	13.7	11.3	18.5	5.08	1793.3	2030.95	1.1325
ISHB450	87.2	111.14	450	250	13.7	9.8	18.78	5.18	1742.7	1955.03	1.1218
ISLB500	75	95.5	500	180	14.1	9.2	20.1	3.34	1543.2	1773.67	1.1493
ISWB450	79.4	101.15	450	200	15.4	9.2	18.63	4.11	1558.1	1760.59	1.13
ISHB400	82.2	104.66	400	250	12.7	10.6	16.61	5.16	1444.2	1626.36	1.1261
ISHB400	77.4	98.66	400	250	12.7	9.1	16.87	5.26	1404.2	1556.33	1.1155
ISMB450	72.4	92.27	450	150	17.4	9.4	18.15	3.01	1350.7	1533.36	1.15
ISLB450	65.3	83.14	450	170	13.4	8.6	18.2	3.2	1223.8	1401.35	1.1451
ISWB400	66.7	85.01	400	200	13	8.6	16.6	4.04	1171.3	1290.19	1.1271
ISHB350	72.4	92.21	350	250	11.6	10.1	14.65	5.22	1131.6	1268.69	1.1212
ISHB350	67.4	85.91	350	250	11.6	8.3	14.93	5.34	1094.8	1213.53	1.1085
ISMB400	61.6	78.46	400	140	16	8.9	16.15	2.82	1022.9	1176.18	1.1498
ISLB400	56.9	72.43	400	165	12.5	8	16.33	3.15	965.3	1099.45	1.139
ISWB350	56.9	72.5	350	200	11.4	8	14.63	4.03	887	995.49	1.1223
ISHB300	63	80.25	300	250	10.6	9.4	12.7	5.29	863.3	962.18	1.1145
ISHB300	58.8	74.85	300	250	10.6	7.6	12.95	5.41	836.3	921.68	1.1021
ISMC400	49.4	62.93	400	100	15.3	8.6	15.48	2.83	754.1	891.03	1.1816
ISMB350	52.4	66.71	350	140	14.2	8.1	14.29	2.84	778.9	889.57	1.1421
ISLB350	49.5	63.01	350	165	11.4	7.4	14.45	3.17	751.9	851.11	1.132
ISLC400	45.7	58.25	400	100	14	8	15.5	2.81	699.5	825.02	1.1794
ISWB300	48.1	61.33	300	200	10	7.4	12.66	4.02	654.8	731.21	1.1167
ISHB250	54.7	69.71	250	250	9.7	8.8	10.7	5.37	638.7	708.43	1.1092

ISLB325	43.1	54.9	325	165	9.8	7	13.41	3.05	607.7	687.76	1.1317
ISHB250	51	64.96	250	250	9.7	6.9	10.91	5.49	618.9	678.73	1.0967
ISMC350	42.1	53.66	350	100	13.5	8.1	13.66	2.83	571.9	672.19	1.1754
ISMB300	44.2	56.26	300	140	12.4	7.5	12.37	2.84	573.6	651.74	1.1362
ISLC350	38.8	49.47	350	100	12.5	7.4	13.72	2.82	532.1	622.95	1.1707
ISLB300	37.7	48.08	300	150	9.4	6.7	12.35	2.8	488.9	554.32	1.1338
ISHB225	46.8	59.66	225	225	9.1	8.6	9.58	4.84	487	542.22	1.1134
ISWB250	40.9	52.05	250	200	9	6.7	10.69	4.06	475.4	527.57	1.1097
ISHB225	43.1	54.94	225	225	9.1	6.5	9.8	4.96	469.3	515.82	1.0987
ISMC300	35.8	45.64	300	90	13.6	7.6	11.81	2.61	424.2	496.77	1.1711
ISMB250	37.3	47.55	250	125	12.5	6.9	10.39	2.65	410.5	465.71	1.1345
ISLC300	33.1	42.11	300	100	11.6	6.7	11.98	2.87	403.2	466.73	1.1576
ISLB275	33	42.02	275	140	8.8	6.4	11.31	2.61	392.4	443.09	1.1305
ISHB200	40	50.94	200	200	9	7.8	8.55	4.42	372.2	414.23	1.1129
ISHB200	37.3	47.54	200	200	9	6.1	8.71	4.51	360.8	397.23	1.101
ISWB225	33.9	43.24	225	150	9.9	6.4	9.52	3.22	348.5	389.93	1.1189
ISMC250	30.4	38.67	250	80	14.1	7.1	9.94	2.38	305.3	356.72	1.1684
ISMB225	31.2	39.72	225	110	11.8	6.5	9.31	2.34	305.9	348.27	1.1385
ISLB250	27.9	35.53	250	125	8.2	6.1	10.23	2.33	297.4	338.69	1.1388
ISLC250	28	35.65	250	100	10.7	6.1	10.17	2.89	295	338.11	1.1462
ISWB200	28.8	36.71	200	140	9	6.1	8.46	2.99	262.5	293.99	1.12
ISMC225	25.9	33.01	225	80	12.4	6.4	9.03	2.38	239.5	277.93	1.1605
ISLC225	24	30.53	225	90	10.2	5.8	9.14	2.62	226.5	260.13	1.1485
ISLB225	23.5	29.92	225	100	8.6	5.8	9.15	1.94	222.4	254.72	1.1453
ISMB200	25.4	32.33	200	100	10.8	5.7	8.32	2.15	223.5	253.86	1.1358
ISHB150	34.6	44.08	150	150	9	11.8	6.09	3.35	218.1	251.64	1.1538
ISHB150	30.6	38.98	150	150	9	8.4	6.29	3.44	205.3	232.52	1.1326
ISHB150	27.1	34.48	150	150	9	5.4	6.5	3.54	194.1	215.64	1.111
ISMC 200	22.1	28.21	200	75	11.4	6.1	8.03	2.23	181.9	211.25	1.1614
ISLC200	20.6	26.22	200	75	10.8	5.5	8.11	2.37	172.6	198.77	1.1516
ISWB175	22.1	28.11	175	125	7.4	5.8	7.33	2.59	172.5	194.2	1.1258
ISLB200	19.8	25.27	200	100	7.3	5.4	8.19	2.13	169.7	184.34	1.137
ISMB175	19.3	24.62	175	90	8.6	5.5	7.19	1.86	145.4	166.08	1.1422
ISMC175	19.1	24.38	175	75	10.2	5.7	7.08	2.23	139.8	161.65	1.1563
ISLC175	17.6	22.4	175	75	9.5	5.1	7.16	2.38	131.3	150.36	1.1452
ISLB175	16.7	21.3	175	90	6.9	5.1	7.17	1.93	125.3	143.3	1.1437
ISJB225	12.8	16.28	225	80	5	3.7	8.97	1.58	116.3	134.15	1.1535
ISJC200	13.9	13.9	200	70	7.1	4.1	8.08	2.18	116.1	133.12	1.1465
ISWB150	17	21.67	150	100	7	5.4	6.22	2.09	111.9	126.86	1.1337
ISMC150	16.4	20.88	150	75	9	5.4	6.11	2.21	103.9	119.82	1.1533
ISMB150	14.9	19	150	80	7.6	4.8	6.18	1.66	96.9	110.48	1.1401

ISLC150	14.4	18.36	150	75	7.8	4.8	6.16	2.37	93	106.17	1.1416
ISLB150	14.2	18.08	150	80	6.8	4.8	6.17	1.75	91.8	104.5	1.1384
ISJC175	11.2	14.24	175	60	6.9	3.6	7.11	1.88	82.3	94.22	1.1449
ISJB200	9.9	12.64	200	60	5	3.4	7.86	1.17	78.1	90.89	1.1639
ISMB125	13	16.6	125	75	7.6	4.4	5.2	1.62	71.8	81.85	1.1399
ISMC125	12.7	16.19	125	65	8.1	5	5.07	1.92	66.6	77.15	1.1585
ISLB125	11.9	15.12	125	75	6.5	4.4	5.19	1.69	65.1	73.93	1.1356
ISJC150	9.9	12.65	150	55	6.9	3.6	6.9	1.73	62.8	72.04	1.1472
ISLC125	10.7	13.67	125	65	6.6	4.4	5.11	2.05	57.1	65.45	1.1462
ISMB100	11.5	14.6	100	75	7.2	4	4.2	1.67	51.5	58.65	1.1389

APPENDIX B



Basic wind speed in m/s

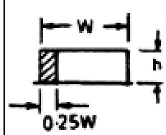
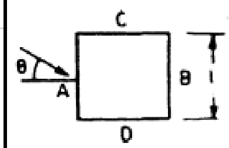
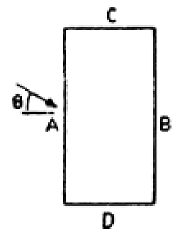
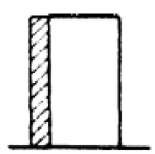
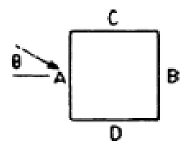
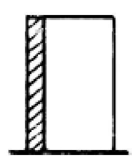
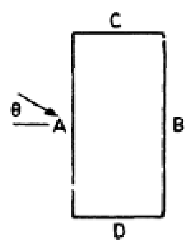
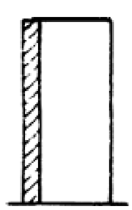
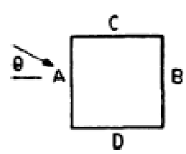
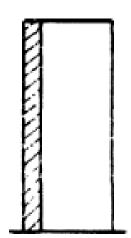
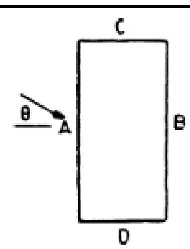
Table 2. Risk coefficients for different classes of structures in different wind speed zones

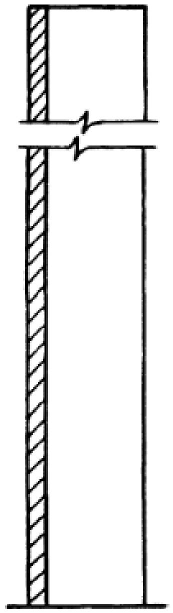
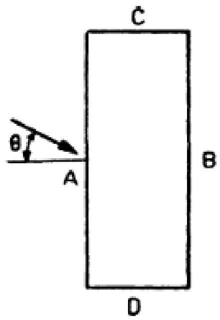
Class of Structure	**k_1 factor for Basic Wind Speed (m/s) of**	**k_1 factor for Basic Wind Speed (m/s) of**					
		33	39	45	47	50	55
All general buildings and structures	50	1.0	1.0	1.0	1.0	1.0	1.0
Temporary sheds, structures such as those used during construction operations (for example, formwork and false work), structures during construction stages, and boundary walls	5	0.82	0.76	0.73	0.71	0.70	0.67
Buildings and structures presenting a low degree of hazard to life and property in the event of failure, such as isolated towers in wooded areas, farm buildings other than residential buildings, etc.	25	0.94	0.92	0.91	0.90	0.90	0.89
Important buildings and structures such as hospitals, communication buildings, towers and power plant structures	100	1.05	1.06	1.07	1.07	1.08	1.08

Table 3. k_2 factors to obtain design wind speed variation with height in different terrains

Height (*z*) *(m)*	Terrain and height multiplier (k_2)			
	Terrain Category 1	**Terrain Category 2**	**Terrain Category 3**	**Terrain Category 4**
10	1.05	1.00	0.91	0.80
15	1.09	1.05	0.97	0.80
20	1.12	1.07	1.01	0.80
30	1.15	1.12	1.06	0.97
50	1.20	1.17	1.12	1.10
100	1.26	1.24	1.20	1.20
150	1.30	1.28	1.24	1.24
200	1.32	1.30	1.27	1.27
250	1.34	1.32	1.29	1.28
300	1.35	1.34	1.31	1.30
350	1.37	1.36	1.32	1.31
400	1.38	1.37	1.34	1.32
450	1.39	1.38	1.35	1.33
500	1.40	1.39	1.36	1.34
NOTE: For intermediate values of height z and terrain category, use linear interpolation.				

Table 4. External Pressure Coefficients (*C*pe) For Walls Of Rectangular Clad Buildings

BUILDING HEIGHT RATIO	BUILDING PLAN RATIO	ELEVATION	PLAN	WIND ANGLE θ	C_{pe} FOR SURFACE *A*	*B*	*C*	*D*	LOCAL C_{pe}
$\frac{h}{w} \leqslant \frac{1}{2}$	$1 < \frac{l}{w} \leqslant \frac{3}{2}$			degrees 0	+ 0.7	− 0.2	− 0.5	− 0.5	− 0.8
				90	− 0.5	− 0.5	+ 0.7	− 0.2	
	$\frac{3}{2} < \frac{l}{w} < 4$			0	+ 0.7	− 0.25	− 0.6	− 0.6	− 1.0
				90	− 0.5	− 0.5	+ 0.7	− 0.1	
$\frac{1}{2} < \frac{h}{w} \leqslant \frac{3}{2}$	$1 \leqslant \frac{l}{w} \leqslant \frac{3}{2}$			0	+ 0.7	− 0.25	− 0.6	− 0.6	− 1.1
				90	− 0.6	− 0.6	+ 0.7	− 0.25	
	$\frac{3}{2} \leqslant \frac{l}{w} < 4$			0	+ 0.7	− 0.3	− 0.7	− 0.7	− 1.1
				90	− 0.5	− 0.5	+ 0.7	− 0.1	
$\frac{3}{2} < \frac{h}{w} < 6$	$1 < \frac{l}{w} \leqslant \frac{3}{2}$			0	+ 0.8	− 0.25	− 0.8	− 0.8	− 1.2
				90	− 0.8	− 0.8	+ 0.8	− 0.25	
	$\frac{3}{2} \leqslant \frac{l}{w} < 4$			0	+ 0.7	− 0.4	− 0.7	− 0.7	− 1.2
				90	− 0.5	− 0.5	+ 0.8	− 0.1	

Building Height Ratio	Building Plan Ratio	Elevation	Plan	Wind Angle θ	C_{pe} for Surface A	B	C	D	Local C_{pe} 
$\frac{h}{w} \geq 6$	$\frac{l}{w} = \frac{3}{2}$			0	+ 0.95	– 1.85	– 0.9	– 0.9	– 1.25
				90	– 0.8	– 0.8	+ 0.9	– 0.85	
	$\frac{l}{w} = 1.0$			0	+ 0.95	– 1.25	– 0.7	– 0.7	– 1.25
				90	– 0.7	– 0.7	+ 0.95	– 1.25	
	$\frac{l}{w} = 2$			0	+ 0.85	– 0.75	– 0.75	– 0.75	– 1.25
				90	– 0.75	– 0.75	+ 0.85	– 0.75	

APPENDIX C

Table 5. Steel sections provided to each beam in STAADPro analysis

Beam No.	Profile	Beam No.	Profile	Beam No.	Profile
1203	ISA100X100X8	1864	ISA130X130X8	1742	ISWB600A
1204	ISA100X100X8	1899	ISWB600	1642	ISA75X75X6
1386	ISA100X100X8	1901	ISWB600A	1641	ISA130X130X12
1388	ISA100X100X8	1903	ISHB250	1389	ISA130X130X12
1385	ISA100X100X8	1792	ISHB150A	1390	ISA130X130X12
1387	ISA100X100X8	1863	ISA130X130X8	1643	ISA75X75X6
224	ISMB600	1862	ISA130X130X8	1640	ISA130X130X12
1895	ISMB250	1645	ISWB600A	1736	ISA130X130X10
1898	ISMB250	1891	ISA130X130X8	1735	ISA130X130X10
1797	ISMB350	1890	ISA130X130X8	1803	ISHB200
1798	ISMB350	747	ISMB350	1837	ISHB150
1394	250X250	743	ISMB350	1970	ISMB100
1393	ISA130X130X12	2021	ISWB250	1813	ISHB150
1392	ISA130X130X12	2018	ISWB175	1917	ISWB175
36	ISWB600A	1749	ISA130X130X10	1748	ISA75X75X6
1644	ISWB600A	1646	ISWB600A	1930	ISHB225
1651	ISWB600A	1751	ISA130X130X10	1927	ISHB150
473	ISMB450	1752	ISA130X130X10	209	ISWB600A
469	ISMB450	1846	ISHB200	1794	ISHB150A
1786	ISMB250	1655	ISWB600	1838	ISWB175
1796	ISMB250	1647	ISWB600A	1963	ISWB175
751	ISMB350	1657	ISWB600A	110	ISHB250
183	ISMB350	1649	ISWB600A	1951	ISWB500
754	ISMB350	81	220X811	1982	ISA130X130X10
443	ISMB250	108	320X900	1669	ISMB350
78	ISMB350	1808	ISHB200	2012	ISHB150
1740	ISMB250	1896	ISWB600A	2037	ISMB100
1784	ISWB600A	213	ISWB600A	2024	ISLB200
1860	ISA200X150X10	73	ISWB600A	37	ISWB600A
1861	ISA200X150X10	1807	ISWB600A	1731	ISWB600
35	ISWB600A	1968	ISLB175	2031	ISA130X130X10
56	ISMB500	1818	ISHB150	2030	ISA130X130X10
58	ISMB500	1932	ISHB150	459	ISMB200
16	ISMB350	1847	ISHB250	1672	ISMB350
14	ISMB350	1840	ISWB175	1673	ISMB350
1911	ISHB150	1921	ISWB175	1648	ISWB600A
2039	ISA100X100X8	1738	ISA130X130X10	1730	ISWB600A
2038	ISA100X100X8	1737	ISA130X130X10	1671	ISMB200
1819	ISHB150	1823	ISHB150	186	ISMB250
1865	ISA130X130X8	235	ISWB600A	109	320X900

Beam No.	Profile	Beam No.	Profile	Beam No.	Profile
82	220X811	1868	ISA55X55X5	567	ISMB500
1652	ISWB600A	1869	ISA55X55X5	1833	ISHB200
1984	ISA130X130X10	1872	ISA55X55X5	107	ISMB200
1940	ISA130X130X10	1876	ISA55X55X5	2010	ISMB100
1941	ISA130X130X10	1877	ISA55X55X5	2013	ISJB225
1983	ISA130X130X10	1873	ISA55X55X5	511	ISMB500
484	ISMB350	2036	ISMB100	1782	ISWB250
1775	ISA130X130X10	2035	ISJB225	1980	ISJB225
2032	ISA130X130X10	755	ISMB350	2016	ISWB175
2033	ISA130X130X10	756	ISMB350	2017	ISHB350
1956	ISLB175	1874	ISA55X55X5	208	ISMB200
1953	ISJB225	1875	ISA55X55X5	303	ISMB250
1947	ISLB175	1885	ISA70X70X5	1978	ISLB175
1912	ISHB200	1884	ISA55X55X5	237	ISMB200
1989	ISA130X130X10	1991	ISA130X130X10	1929	ISHB200
1990	ISA80X80X6	1992	ISA130X130X10	1928	ISHB200
1939	ISA130X130X10	2000	ISLB75	1926	ISHB200
1938	ISA130X130X10	2003	ISLB75	1925	ISHB200
2020	ISMB100	1969	ISJB225	1852	ISJB225
2023	ISJB225	1958	ISLB175	1848	ISMB600
1996	ISA130X130X10	1993	ISJB225	1814	ISJB225
1997	ISA130X130X10	1962	ISJB225	1849	ISJB225
1888	ISA75X75X5	1960	ISLB175	1806	ISMB600
1889	ISA70X70X5	1920	ISHB200	1821	ISJB225
1986	ISA75X75X5	1904	ISHB150	1855	ISJB225
1935	ISA130X130X10	1924	ISHB200	1854	ISMB600
185	ISMB250	1750	ISA130X130X10	1820	ISMB600
1942	ISWB600A	2014	ISLB400	1795	ISWB600A
1653	ISWB600	2015	ISHB150	1793	ISWB600
1900	ISHB200	2025	ISLB200	1894	ISWB600A
2028	ISA130X130X10	1781	ISLB400	1713	ISMB350
2029	ISA130X130X10	2007	ISHB150	742	ISMB450
2005	ISHB350	2019	ISLB200	757	ISMB200
2027	ISHB350	1785	ISWB600A	746	ISMB450
1879	ISA60X60X5	1961	ISWB175	1714	ISMB350
1878	ISA60X60X5	1954	ISWB500	77	ISMB450
2034	ISA130X130X10	1923	ISLB75	80	ISMB350
1918	ISHB200	1937	ISLB75	1716	ISMB350
1922	ISHB200	1880	ISHB200	752	ISMB450
1886	ISA70X70X5	1955	ISHB250A	1715	ISMB350
1887	ISA65X65X5	1965	ISWB175	749	ISMB450
1871	ISA55X55X5	1857	ISMB600	1656	ISWB600A

Beam No.	Profile	Beam No.	Profile	Beam No.	Profile
1791	ISWB600A	2004	ISHB150	1910	ISHB200
1654	ISWB600	1829	ISHB200	1966	ISLB175
1733	ISWB600A	1843	ISLB100	1805	ISHB150
1987	ISHB250A	1972	ISLB175	1893	ISWB500
605	ISWB600	449	220X750	1976	ISMB250
1988	ISLB100	1660	ISMB600	1952	ISHB250A
1919	ISLB75	478	ISMB200	1776	ISWB600
1831	ISHB200	477	ISMB200	2006	ISWB250
1975	ISLB100	483	ISMB200	1779	ISHB350
1881	ISHB200	479	ISMB200	1998	ISMB100
1936	ISLB75	1945	ISHB250A	1999	ISLB75
1950	ISHB250A	181	220X750	1944	ISWB600
239	ISWB600	1973	ISMB250	1897	ISMB600
1835	ISJB225	1974	ISWB500	1756	ISMB600
1834	ISMB600	1892	ISWB600A	1754	220X750
1995	ISJB225	517	ISMB500	1957	ISHB200
1994	ISJB225	1755	ISMB600		
2026	ISLB400	178	ISWB600A		
2011	ISLB400	1739	ISWB600A		
1883	ISHB150	1741	ISWB600		
1763	ISMB600	1825	ISHB200		
1757	ISHB200	1247	ISMB600		
1882	ISJB225	1674	ISMB350		
1851	ISMB600	1670	ISMB350		
442	220X750	177	ISWB600A		
444	ISMB600	1734	ISWB600A		
2022	ISLB200	1931	ISHB200		
2009	ISHB150	86	ISWB600		
2008	ISLB400	1902	ISHB200		
253	ISMB500	85	ISWB600A		
1959	ISWB175	1769	ISWB600A		
1946	ISWB500	1391	ISWB600A		
1943	ISHB250A	1650	ISWB600A		
1668	220X750	1747	ISWB600A		
447	ISMB600	182	ISMB600		
1836	ISWB175	1666	220X750		
1850	ISLB75	1665	ISMB600		
1853	ISLB75	2001	ISMB100		
1856	ISLB75	2002	ISLB75		
1859	ISLB75	1979	ISLB75		
1905	ISMB100	1967	ISMB100		
1913	ISWB175	238	ISMB500		

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