

ANALYSIS AND DESIGN OF STEEL PIPE RACKS USING TUBULAR SECTIONS

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Abstract: - Apart from the view on delineation for stability or economic or practical aspects of steel structures, the design of pipe supporting structures are commonly overemphasized. This may be sometimes described as over-designing or under-detailing. The structure which transfers the load from the pipe to the supporting structure is commonly referred to as Pipe Supporting Structures. In this paper, the terms pipe racks, pipe supports, and pipe support structures are alike. The pipe rack structural system will be comprised of secondary elements and less impact on the structural integrity of the industrial facility. The failure of pipe rack structures is neither accounted or spreader over the structural community. The structural design of pipe racks is varied accordance to the plant operation and standards. But the failure of pipe rack may cause the serviceability problems to the plant operations. The failures of pipe support system may risk to the health, welfare, and safety of plant personnel due to breakage or leakages in the pipe system. The following discussion includes a review of the considerations involved in the design, detailing, and structural stability of pipe racks. Optimal solutions are still governed by the judgment of design engineer. Pipe rack structures are used extensively throughout industrial facilities worldwide.

Key Words: Piperack, Bracing, Tubular section, Supporting structure

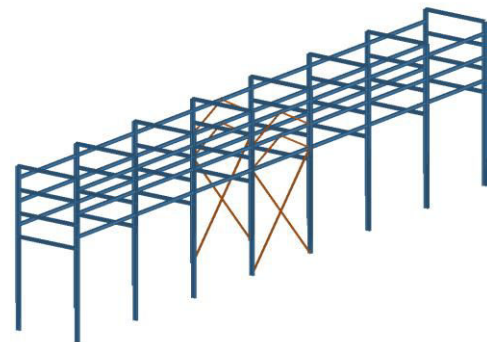
1 INTRODUCTION

1.1 GENERAL

The supporting structure which carries the pipes or cables or conduits systems are referred to as Pipe racks. The pipe rack structures are considered to be as non-building structures. Although it is non-building structures, it still to be designed with the effect of stability analysis. Typical Pipe racks are long lengthened - narrow structures and carries the pipes in the longitudinal direction. Typical Four-Level Pipe Racks Consists of Eight Transverse Frames Connection by Longitudinal Strut. Fig. 1.1 shows a typical pipe rack used in an industrial facility. The transverse frames are required to be designed as moment resisting frames for the purposes of Pipe routing, maintenance access, and access corridors. The moment resisting frames are to be designed against gravity loads as well as lateral loads from either pipe loads or wind and seismic loads. The transverse frames are joined using longitudinal struts with one bay typically braced. For any loads raised longitudinally will be transmitted to the longitudinal struts and carry over to the bracing system.

The pipe rack structures are mainly for the operation of industrial facility but the design and analyses of pipesupporting structures are not usually covered in the code referenced documents. This lack of standard in the design of pipe rack structures leads to each individual or firms adopting its own standards without understanding the design concepts. Process Industry Practices Structural Design Criteria (PIP STC01015) has tried to develop a common standard for design but this is not considered as a code document.

The deficiency of code documents brings to the uncertainty in the design of pipe racks. But the notion on the stability analysis should not be ignored based on the lack of code referenced documents. AISC 360-10 still be used as references for the analysis and design based on the stability.



1.1.1.1 Fig. 1.1 Typical Four-Level Pipe Racks Consisting of Eight Transverse Frames Connection by Longitudinal Struts

There are different types of pipe rack systems available such as

1. Continuous Pipe racks (conventional pipe rack) system,
2. Non-continuous Pipe racks system,
3. Modular Pipe rack.

Continuous pipe rack system is essentially a system where multiple 2-dimensional (2D) frame assemblies (commonly called bents), comprised of two or more columns with transverse beams, are tied together in the longitudinal direction utilizing beam struts (for support of transverse pipe

and raceway elements and for longitudinal stability of the system) and vertical bracing to form a 3D space frame arrangement as shown in Fig. 1.2 (a). Pipe racks supporting equipment such as air-cooled heat exchangers must utilize the continuous system approach.

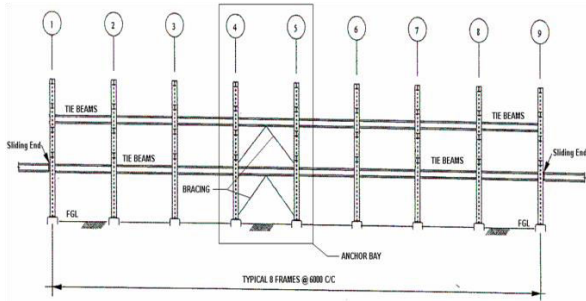


Fig. 1.2 (a) Longitudinal elevation of continuous pipe rack

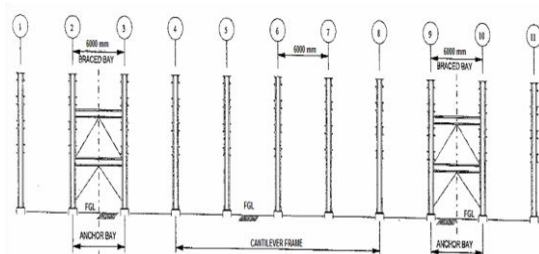


Fig. 1.2 (b) Longitudinal elevation of non-continuous pipe rack

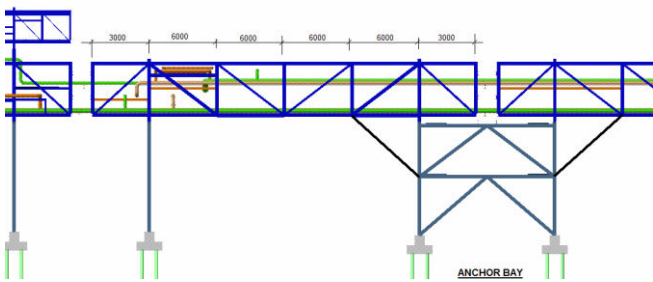


Fig. 1.2 (c) Member unity check

Fig. 1.2 Different types of pipe rack systems

Non-continuous pipe rack system comprises of independent cantilevered, freestanding 2D frames not dependent on longitudinal beam struts for system stability as shown in Fig. 1.2 (b). This system, where feasible, should result in lower total installed cost. **Modular Pipe Racks** is very economical to modularize pipe rack structures located in remote sites with harsh climate conditions. Although pipe rack modularization results in substantial savings to the project cost, the steel quantity may increase by almost 30%. Also, additional cost might result to cover the assembly and transportation procedures. It is the author's opinion that, to minimize construction errors, structural drawings must be

issued for all individual modules detailing the assembly and erection procedures. This requirement might increase the engineering hours, but the resulting cost is incomparable to repairing construction errors. Modularized pipe racks are fabricated off-site as small modules and outfitted with piping, electrical, instrumentation, and mechanical equipment as shown in Fig. 1.2 (c).

A pipe rack is the main artery of a process unit. Pipe racks carry process and utility piping and may also include instrument and cable trays as well as equipment mounted over all of these. The research work has been carried out on worldwide on the behavior of pipe rack structures for the last two decades.

Osama Bedair et al. (2014) carried out an analytical study to give some industrial guidelines for practicing engineers and steel fabricators to design Steel pipe racks. Currently there is no given procedure for the standardization to implement. In practice, there is currently no standard procedure to implement. Unfortunately, the design is performed in a roandom fashion, and the Engineers may overlook critical aspects. The author was involved in several megaprojects during the feasibility and detailed design phases and will present to readers some critical aspects for the design of pipe racks. Additionally, there is a focus on the design coordination required by engineering disciplines. The paper also describes modeling of the pipe rack design loads, modularization procedure, and the foundation system. The author found that essential Design aspects are sometimes overlooked, and existing rules are not Adequate. The available design rules provided by North American and European codes mainly address building designs. This paper highlighted Critical design issues and provided recommendations for pipe Rack designs used in oil sands and petrochemical facilities.

Drake et al. (2012) concluded that the requirements found in the building codes apply and dictate some of the design requirements. Some code requirements are not clear on how they are to be applied to pipe racks, because most are written for buildings. Several industry references exist to help the designer apply the intent of the code and follow expected engineering practices. Engineering practices vary and are, at times, influenced by client requirements and regional practices. Additional and updated design guides are needed so that consistent design methods are used throughout the industry.

Bendapudi et al. (2010) concluded that lack of uniform industry standards for this topic leads to each organization adopting its own engineering standards, at times, without a clear understanding of the underlying theoretical concepts and the cost implications. This is the first of a two-part series of articles on the behavior and design of steel support structures for pipes. This article discusses the effects of

ambient temperature changes, expansion joint requirements, and an introduction to design loads. Part 2 concludes with the continuation of design loads, structure stability concepts and detailing for stability requirements. It is common to overemphasize the structural design of pipe support structures, rather than focus on detailing for stability or economics and practical aspects of the steel structure and the foundations. This is sometimes referred to as "over-designing" and "under-detailing". Sometimes the hanger-type pipe supports or the trapezes supported by another structure, such as the main building frame, are referred to as "pipe support structures."

Nelson et al. (2008) carried out analytic study of steel pipe racks for stability criteria. Pipe rack structures are used extensively throughout industrial facilities worldwide. While stability analysis is required in pipe rack design per the AISC Specification for Structural Steel Buildings (AISC 360-10). The most compelling reason for uniform application of stability analysis is more fundamental. Improper application of stability analysis methods could lead to unconservative results and potential instability in the structure jeopardizing the safety of not only the pipe rack structure but the entire industrial facility. The direct analysis method, effective length method and first order method are methods of stability analysis that are specified by AISC 360-10. This tendency for large second order effects demands careful attention in stability analysis. Proper application as well as clear understanding of the limitations of each method is crucial for accurate pipe rack design.

Akbar Shahiditabar et al. (2013) identified ambiguities and problems in the pipe and pipe rack design method due to not modeling pipe in pipe rack design. The experienced damages in previous earthquakes confirm the mentioned claim. This study aims to propose a new method for considering pipe and pipe rack interaction instead of current method in order to solve current problems. In the proposed method, pipe is framed to pipe rack in all points and then pipe and supporting structure are design simultaneously. The proposed method is assessed by modeling in Sap and Caesar programs with nonlinear static analysis which results confirm our claims. In our suggested method, current problems are solved and the amount of used materials is reduced up to 29%.

Kalyanshetti et al. (2012) concluded that most of the steel structures are built-up with conventional sections of steels which are designed and constructed by conventional methods. This leads to heavy or uneconomical structures. Tubular steel sections are the best replacements to the conventional ones with their useful and comparatively better properties. It is obvious that due to the profile of the tube section, dead weight is likely to be reduced for many structural members which derives overall economy. This study is regarding the economy, load carrying capacity of all

structural members and their corresponding safety measures. Economy is the main objective of this study involving comparison of conventional sectioned structures with Tubular sectioned structure for given requirements. For study purpose superstructure-part of an industrial building is considered and comparison is made. Study reveals that, upto 40 to 50% saving in cost is achieved by using Tubular sections.

Mohammad Karimi et al. (2011) studied the safe and stable production process. The behavior of these supporting structures is similar to steel or reinforced concrete frame supporters for elevated processing pipes. Qualitative and quantitative methods of seismic vulnerability evaluations have been used according to the ASCE-1998 standards. Computer modelings have been used in quantitative evaluation of the supporting structures, including equivalent Static analysis and linear dynamic analysis by considering torsion and P- Δ effects. Also, gravity and thermal loads based on the existing documents and design calculation sheets and specification notes have been considered in the analyses. Gravity and lateral load combinations have been considered for seismic evaluation of foundation systems. Overturning stability of structures and uplifting of foundation systems due to the gravity and lateral loads, and also, lateral displacements, frame element and connection capacities have been investigated. However, different methods of seismic strengthening and retrofitting of structural system have been proposed.

Mallikarjuna et al. (2014) concluded that the high rise buildings require high frame structure stability for safety and design purposes. This paper focused on P-delta analysis to be compared with linear static analysis. In this study, an 18 storey steel frame structure with 68.9 m has been selected to be idealized as multi storey steel building model. The model is analyzed by using STAAD.Pro 2007 structural analysis software with the consideration of P-delta effect. At the same time the influence of different bracing patterns has been investigated. For this reason five types of bracing systems including X, V, Single Diagonal, Double X, K bracing with unbraced model of same configuration are modeled and analyzed. The framed structure is analyzed for Wind load as per IS 875 (part 3)-1987. After analysis, the comparative study is presented with respect to Maximum storey displacement and Axial Force. The present work showed that the 'X' bracing in continuous bracing pattern is proved to be more effective with respect to both Static and P-delta analysis.

Jagadish et al. (2013) studied the effect of different types of bracing systems in multi storied steel buildings. For this purpose the G+15 stories steel building models is used with same configuration and different bracing systems such as Single-Diagonal, X bracing, Double X bracing, K bracing, V bracing is used. A commercial software package STAAD.Pro

V8i is used for the analysis of steel buildings and different parameters are compared.

Siddiqi et al. (2014) concluded that lateral stiffness is a major consideration in the design of tall buildings. Bracing is a highly efficient and economical method of resisting lateral forces in a frame structure because the diagonals work in axial stress and therefore call for minimum member sizes in providing the stiffness and strength against horizontal shear. He has taken five different types of bracing systems for the use in tall building in order to provide lateral stiffness and finally the optimized design in terms of lesser structural weight and lesser lateral displacement has been exposed. For this purpose a sixty storey regular shaped building is selected and analyzed for wind and gravity load combinations along both major and minor axes.

Ziaulla Khan et al. (2015) studied the effect of four different types of bracing systems for the use in SMRF RC framed building situated in seismic zone IV, in order to provide lateral stiffness and results in terms of storey shears and storey drifts have been discussed. He studied the seismic behavior of RC building by performing linear static and non linear static analysis & Comparative study for concentrically and eccentrically placed lateral load resisting systems at different locations is performed using FEM based analytical software ETABS 9.7.4. He compared various parametric results such as Storey drift and Storey forces for the different models. To obtain pushover curves both in X and Y directions using FEM based analytical software ETABS 9.7.4 is used.

ZasiahTafheem et al. (2013) carried out analytic study of performance of steel building with different types of bracing systems such as concentric(crossed X) bracing and eccentric (V-type) bracing using HSS sections. The performance of the building has been evaluated in terms of lateral storey displacement, storeydrift as well as axial force and bending moment in columns at different storey level. The effectiveness of various types of steel bracing on the structure has also been investigated. More importantly, the reduction in lateral displacement has been found out for different types of bracing system in comparison to building with no bracing.

1.2 OBJECTIVE

The main objective is to carry out a cost benefit study to analyze and design steel pipe racks using tubular sections and to compare the different types of bracing systems of steel pipe racks. To ensure safety of members against lateral loads by carry out explicit stability analysis.

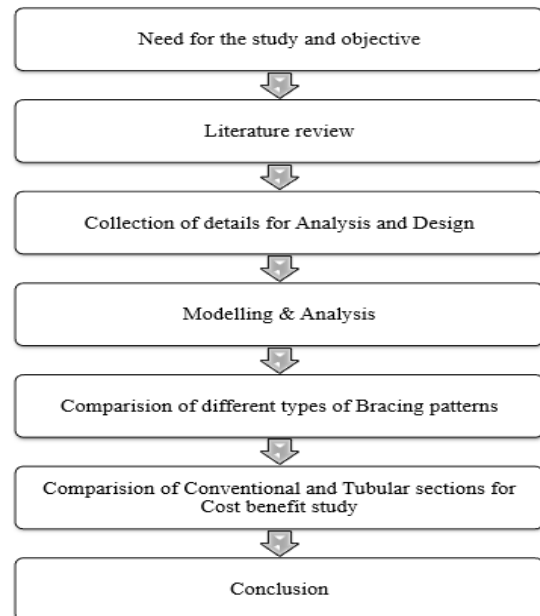
1.3 NEED FOR THE STUDY

In existing steel pipe rack structure, I – sections and built up steel sections are commonly used. Considering the economic factors, there is a need to reduce the steel

consumption in order to bring down the construction costs. Hollow sections are known to possess high torsional capacity. Hence there is a need to quantify the amount of savings that can be achieved by using tubular sections.

1.4 METHODOLOGY

The methodology to be adopted for the project work is shown in Fig. 1.3



1.4.1.1 Fig. 1.3 Methodology

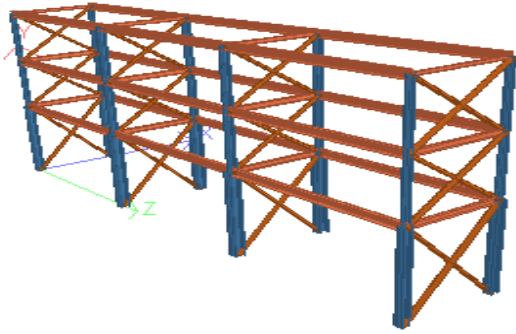
2 PIPE RACK SYSTEM - ANALYSIS

2.1 GENERAL

A conventional continuous pipe rack system has been taken for this study. The details of the system, the load calculations and the analysis details has been done.

2.2 LAYOUT OF PIPE RACK SYSTEM

An elevated multi-level pipe rack may be required for plant layout, equipment or process reasons. Multiple levels are not mandatory; it is simply a question of space. As long as the required space beneath the pipe rack for accessibility and road crossings has been taken into account, the rack can remain single level. However, in most cases, multiple levels will be required. Within plant units, most process pipes are connected to related unit equipment. Placing these pipes in the lower levels results in shorter pipe runs, savings on piping costs and better process flow conditions. Fig 2.1 shows the configuration used in the analysis of pipe racks.



2.2.1.1 Fig 2.1 Steel pipe rack model

The properties of the sections used for the pipe rack system is shown in Table 2.1.

2.2.1.2

2.2.1.3 Table 2.1 Properties of sections

Beams	ISLC100,175,250 ISM175,ISJC150
Columns	ISHB225,250(Ground floor column) ISSC120,ISHB250(1 st & 2 nd floor column)
Bracings	ISA110x110x8

2.3 DESIGN LOADS

Following loads are to be considered for the pipe rack analysis.

2.3.1 Self-Weight of Pipe Rack

The weight of all structural members, including fireproofing, should be considered in the design of the pipe rack.

2.3.2 Piping Gravity Load

In the absence of defined piping loads and locations, an assumed minimum uniform pipe load of 2.0 kPa should be used for preliminary design of pipe racks. This corresponds to an equivalent load of 6 in (150 mm) lines full of water covered with 2" (50 mm) thick insulation, and spaced on 12" (300 mm) centers.

2.3.3 Electrical Tray and Conduits

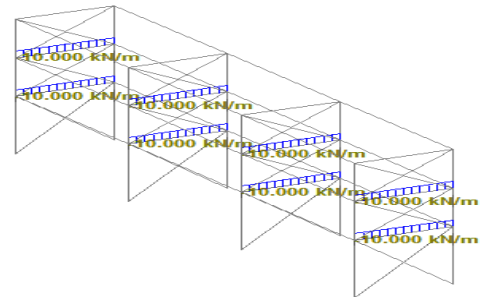
Unless the weight of the loaded raceways can be defined, an assumed minimum uniform load of 1.0 kPa should be used for single tier raceways.

2.3.4 Weight of Equipment on Pipe Rack

Equipment weights, including erection, empty, operating, and test (if the equipment is to be hydro-tested on the pipe rack), should be obtained from the vendor drawings. The equipment weight should include the dead weight of all associated platforms, ladders, and walkways, as applicable. Fig 2.2 shows the dead load input given in STAAD Pro based on all the above-mentioned considerations.

Total pressure (dead load) on the pipe rack = 4kPa

Total udl on the pipe rack = 4x2.5=10kN/m



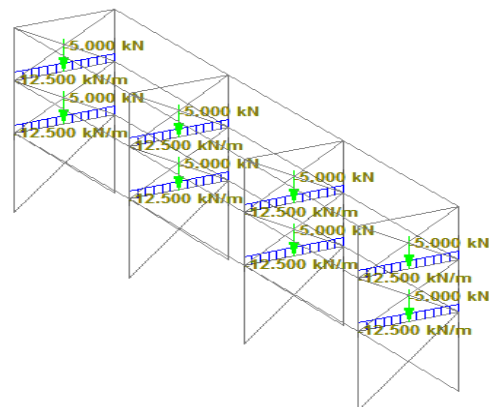
2.3.4.1 Fig 2.2 Dead load

2.3.5 Live Load

A minimum product load of 5kPa shall be used at each Level for the design of major pipe racks. This is equivalent to 8" (203 mm) pipes full of water spaced at 15" (381 mm) centers. Fig 2.3 shows the live load input given in STAAD Pro.

Total pressure (live load) on the pipe rack= 5kPa

Total udl on the pipe rack = 5x2.5=12.5kN/m



2.3.5.1 Fig 2.3 Live load

2.3.6 Wind Load

Transverse wind load on structural members, piping, electrical trays, equipment, platforms, and ladders should be determined in accordance with project approved design code. Longitudinal wind should typically be applied to structural framing, cable tray vertical drop (if any), large dia pipes

vertical drop (if any) and equipment only. The effects of longitudinal wind on piping and trays running parallel to the wind direction should be neglected. Figure 2.4 show the wind load input given in STAAD Pro.

Location of the pipe rack - Chennai

Basic wind speed - 50m/sec

Topography - Flat

Terrain category - 2

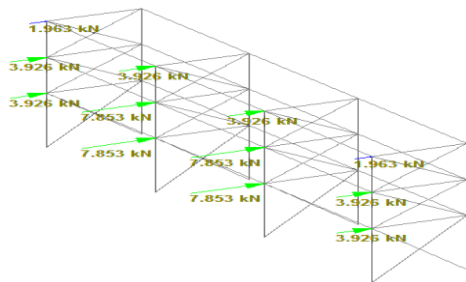


Fig 2.4 (a) Wind load in +X direction

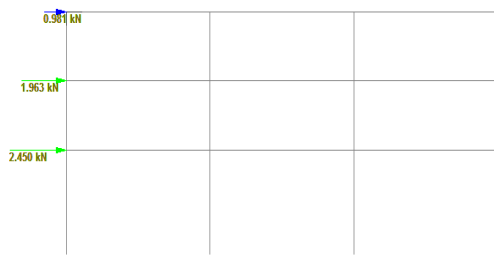
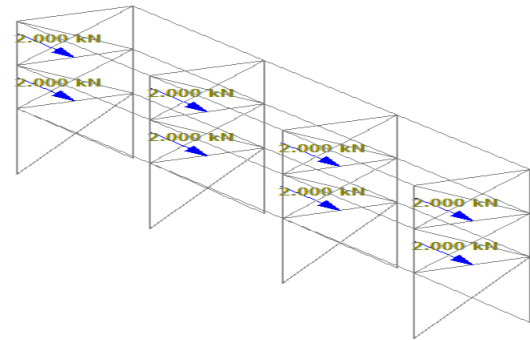


Fig 2.4 (b) Wind load in +Z direction

Fig 2.4 Wind load Input in STAAD Pro

2.3.7 Friction Load

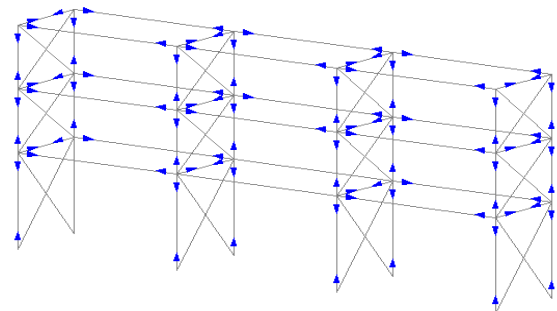
Friction forces caused by hot lines sliding across the pipe support during startup and shutdown are assumed to be partially resisted through friction by nearby cold lines. Therefore, in order to provide for a nominal unbalance of friction forces acting on a pipe support, a resultant longitudinal friction force equal to 7.5% of the total pipe weight or 30% of any one or more lines known to act simultaneously in the same direction, whichever is larger, is assumed for pipe rack design. Friction between piping and supporting steel should not be relied upon to resist wind or seismic loads. Fig 2.5 shows the friction load input given in STAAD Pro.



2.3.7.1 Fig 2.5 Friction load

2.3.8 Temperature Forces

Thermal loads shall be defined as forces caused by changes in the temperature of piping. Pipe supports must be designed to resist longitudinal loads arising from pipe thermal expansion and contraction. These loads are applied to the transverse beams either through friction or through pipe anchors. Thermal loads shall be considered as dead load and included in the appropriate load combinations. Fig 2.6 shows the temperature load input given in STAAD Pro.



2.3.8.1 Fig 2.6 Temperature load

2.3.9 Earthquake Load

Earthquake loads are specified in IS 1893(part-1)2002. Figures 3.8 and 3.9 show the earthquake load input given in STAAD Pro.

Details for the earthquake loading: Location of project - Chennai (zone-3), Type of soil - Medium soil (type-2), Zone factor - 0.16, Importance factor - 1.5, Response reduction factor - 5, Fundamental natural period (as per IS 1893(part-1)2002), $T_a = 0.3658$ Seconds

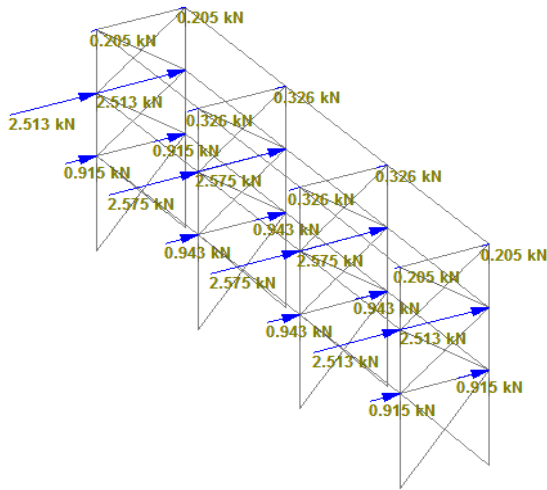


Fig 2.7(a) Earthquake load in X direction

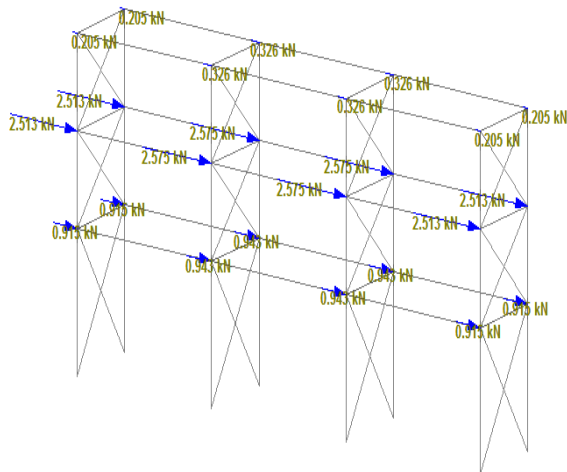


Fig 2.7(b) Earthquake load in Z direction

Fig 2.7 Earthquake load Input in STAAD Pro

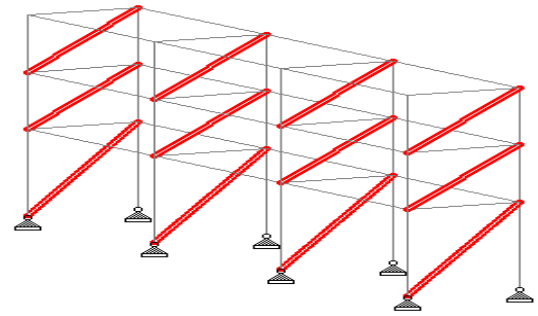


Fig 2.8(a) Single Diagonal Bracing along Transverse and Longitudinal direction

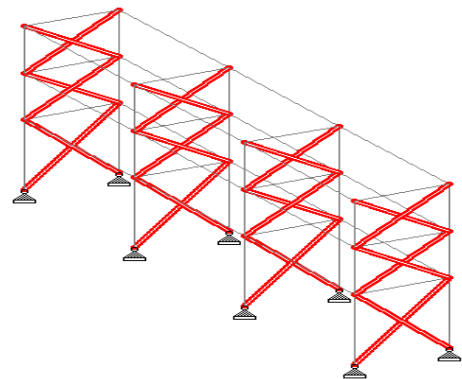


Fig 2.8(b) Double Diagonal Bracing(X bracing) Along Transverse and Longitudinal direction

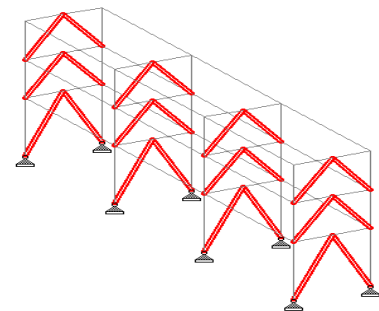


Fig 2.8(c) K Bracing along Transverse and Longitudinal direction

2.4 BRACING CONFIGURATIONS USED IN PIPE RACK

There are different types of bracing systems considered for the analysis of pipe rack structures viz. single diagonal bracing, double diagonal bracing, k/chevron bracing, V bracing. The bracing members were selected automatically using **SELECT OPTIMIZED** option.

The wind loading was considered only in one direction at a time; therefore, two different bracing arrangements were used (i) bracings along longitudinal direction; and (ii) bracings along transverse direction respectively. For both arrangements, the effect of different bracing types was studied. Steel pipe rack models with different options of bracing systems are shown in Fig.2.8.

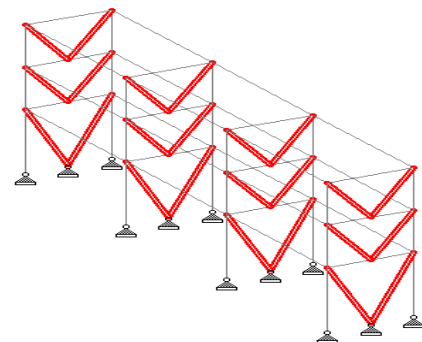


Fig 2.8(d) V Bracing along Transverse and Longitudinal direction

Fig 2.8 Different Configuration of Bracing System

2.5 STABILITY ANALYSIS

Stability analysis is a broad term that covers many aspects of the design process. At present there is no Indian code that lays specifications on stability analysis. Hence the procedure for stability given in AISC has been adopted. According to the 2010 AISC Specification for Structural Steel Buildings (AISC 360-10) stability analysis shall consider the influence of second order effects (P-Δ and P-δ effects), flexural, shear and axial deformations, geometric imperfections, and member stiffness reduction due to residual stresses. Stability analysis is required for all steel structures according to AISC 360-10.

2.5.1 Pipe Rack Model for Stability Analysis

The typical pipe rack was chosen and modeled based on idealized conditions. A width of 2.5m was chosen to allow one-way traffic along the pipe rack corridor. The height of the first level of the pipe rack was set at 3m to provide sufficient height clearance along the access corridor.

The overall length of the pipe rack was set at 15m. Central bays of each segment are typically braced in the longitudinal direction. This allows the length of the pipe rack to expand and contract about a central braced bay and reduces thermally induced loads cause from restraint of thermal movement. If each end of the segment were to consist of a braced frame, the length of the pipe rack would essentially be locked in place and higher thermally induced loads would be seen.

Moment frames are typically spaced at 4-6 meters. This spacing is typically chosen based on the maximum allowable spans for the pipes or cable trays being supported. This spacing can vary based on the estimated size and allowable deflection limits of the pipe being supported. To simplify the design and analysis, a typical moment frame will be selected and isolated for analysis and design

Based on initial calculations that compare the results of the isolated moment frame and the entire pipe rack segment, relatively small differences were seen. Ratios of demand to capacity showed errors of less than 5% on member design when using the single frame compared to the full pipe rack structure. Therefore, analysis of a single moment frame will be used to simplify calculations.

3 RESULTS AND DISCUSSIONS

3.1 GENERAL

To study the behavior of pipe supporting systems a detailed analysis has been carried out using STAAD Pro considering all the load cases elaborately. The details of the analysis carried out on pipe racks fabricated using conventional and tubular section is presented.

3.2 STEEL PIPE RACKS USING CONVENTIONAL SECTIONS

3.2.1 Shear Force Distribution

The shear force distribution is shown in figure 3.1. It can be seen that the maximum shear force occurs due to the load combination (1.5DL+1.5LL) and is found to be 47.142kN. The maximum shear force developed in the beam is less than 0.6 times of its ultimate capacity. i.e. $0.6V_d \geq V_u$ (68.88).

3.2.2 Bending Moment Distribution

The bending moment distribution is shown in figure 3.2. Maximum +ve bending moment occurs due to load combination (1.5DL+1.5LL) and is found to be 18.25kNm. Maximum -ve bending moment is found to be 12.84kNm. It can be seen that external maximum bending moment is less than the maximum resisting moment capacity of the section (34.17 kN-m).

3.2.3 Member Unity Check

The results of the member unity check are shown in figure 3.3. It can be seen that all members are optimally loaded.

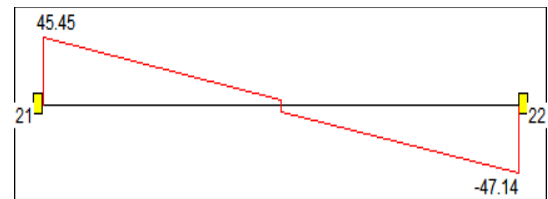


Fig 3.1 Shear force distribution

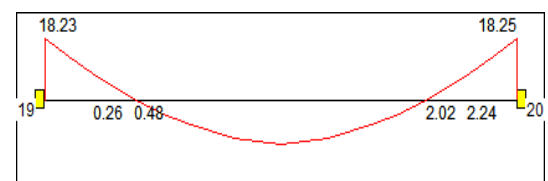


Fig 3.2 Bending moment distribution

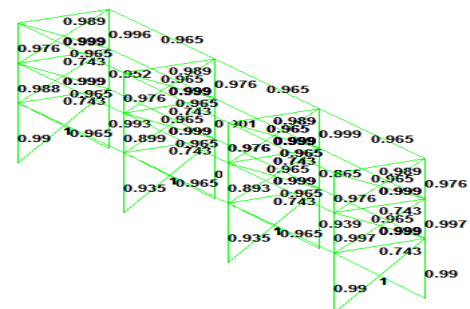


Fig 3.3 Member unity check

3.2.4 Lateral Storey Displacements

Figures 4.4 and 4.5 show the comparison of lateral displacement with permissible limit of lateral displacement which is $H/250$ as per IS 1893-2002, where H is building height in metre. Fig. 3.4 show the lateral displacement obtained with different options of bracing provision and compared with permissible limit of lateral displacement which is $H/500$ as per IS 875(Part 3).

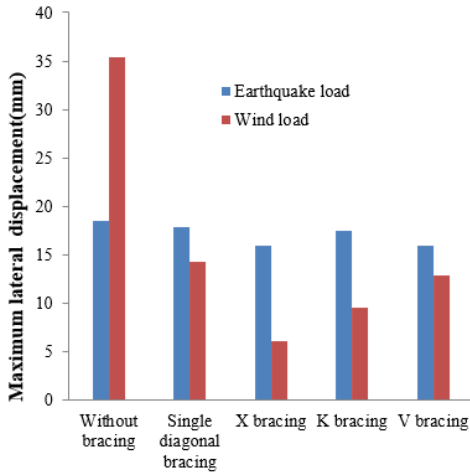


Fig. 3.4 Lateral Displacements along Transverse direction

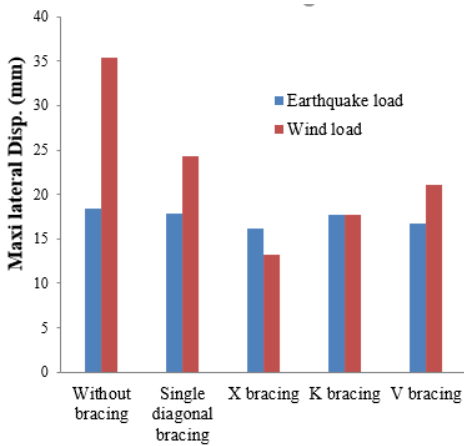


Fig. 3.4 Lateral Displacements along Longitudinal direction

Fig. 3.4 Lateral Displacements

3.2.5 Column Axial Forces and Bending Moments

The variation of maximum axial forces (Compressive) and bending moments for columns due to the combined effect of static and lateral loading has been shown in the following Fig. 3.5a,b c and d.

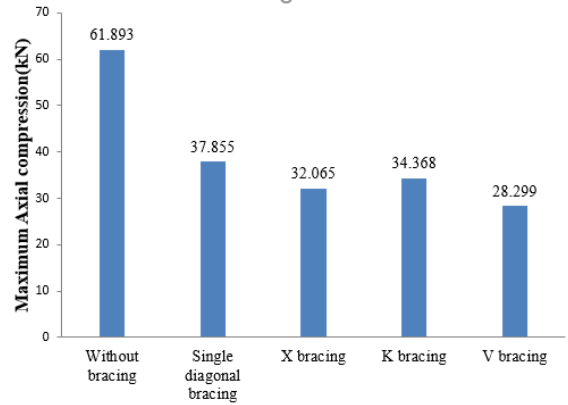


Fig. 3.5(a) Column Axial forces along Transverse direction

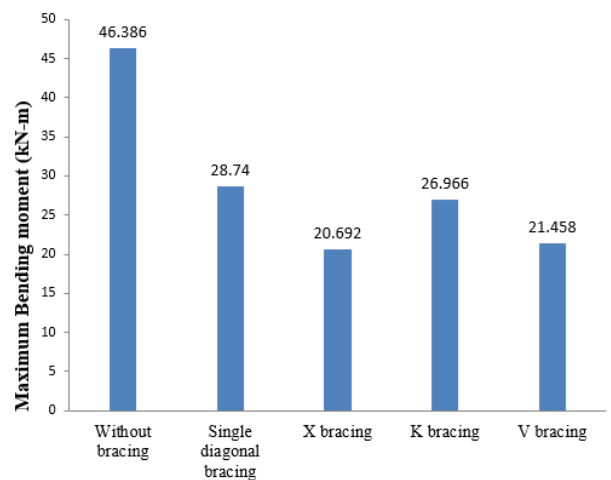


Fig. 3.5(b) Column Bending moment along Transverse direction

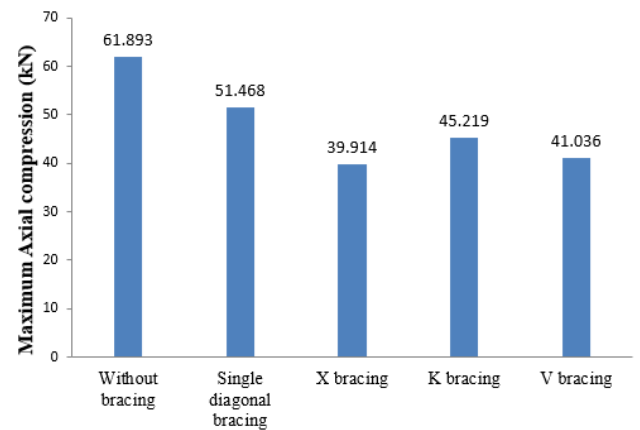


Fig. 3.5(c) Column Axial forces along Longitudinal direction

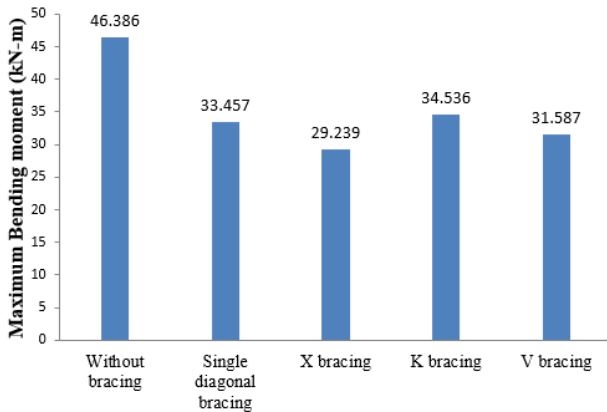


Fig. 3.5(d) Column bending moment along Longitudinal direction

Fig. 3.5 Column Axial Forces and Bending Moments in Longitudinal and Transverse

3.3 STEEL PIPE RACKS USING TUBULAR SECTIONS

The dimension of the tubular sections used is shown in Table 4.7.

Table 4.5 Properties of Tubular sections

Beams	45x45x2.6, 72x72x3.2, 110x110x4.5, 140x80x4.5
Columns	110x110x4.5, 150x150x5 (Ground floor column) 122x61x5.4, 80x40x3.2, 140x80x4.5, 48x48x2.9 (1 st and 2 nd floor column)
Bracings	89x89x3.6, 63x63x3.2

3.3.1

3.3.2 Shear Force Distribution

The shear force distribution diagram is shown in Fig 3.6. It can be seen that the maximum shear force occurs due to load combination 13 (1.5DL+1.5LL) is found to be 46.204kN. The maximum shear force developed in the beam is less than 0.6 times of its ultimate capacity. i.e. $0.6V_d \geq V_u$ (89.4). Hence shear force is not a governing criterion.

3.3.3 Bending Moment Distribution

The bending moment distribution diagram is shown in Fig 3.7. Maximum +ve bending moment occurs due to load combination 13(1.5DL+1.5LL) is found to be 15.4kNm. Maximum -ve bending moment is found to be 15.9kNm. It can be seen that external maximum bending moment is less than the maximum resisting moment capacity ($M_d = \beta b Z_{p_f} / \gamma_{m0} = 17.3$ kNm).

3.3.4 Member Unity Check

The results of the member unity check are shown in Fig 3.8. It can be seen that the slenderness ratio is almost one in all the members indicating optimal member selection.

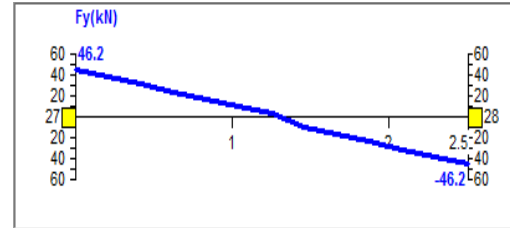


Fig 3.6 Shear force distribution

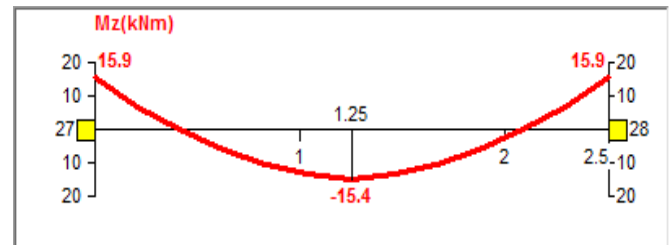


Fig 3.7 Bending moment distribution

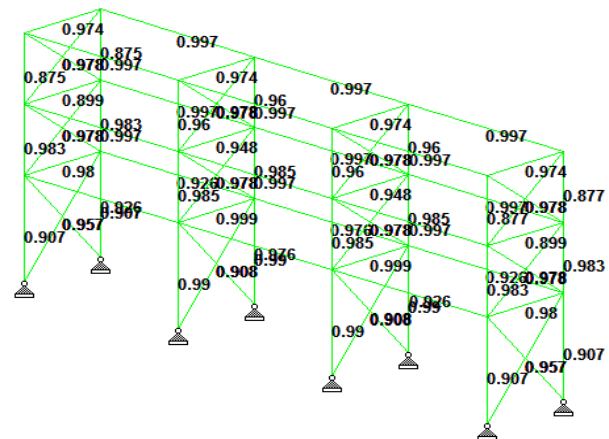


Fig 3.8 Member unity check

3.3.5 Lateral Storey Displacements

Magnitude of lateral displacement induced due to earthquake and wind loads are given in Table 4.8 to 4.11. From the figures 4.15 and 4.16 the values of lateral displacement obtained with different options of bracing provision are also compared with permissible limit of lateral displacement which is $H/250$ as per IS 1893-2002, where H is building height in metre. From the figures 4.17, 4.18 the values of lateral displacement obtained with different options of bracing provision are also compared with permissible limit of lateral displacement which is $H/500$ as per IS 875(Part 3).

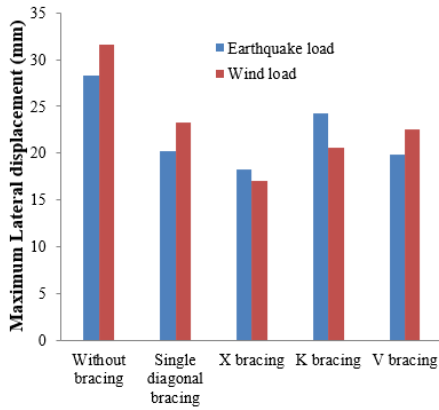


Figure 3.9(a) Lateral Displacements along Transverse direction

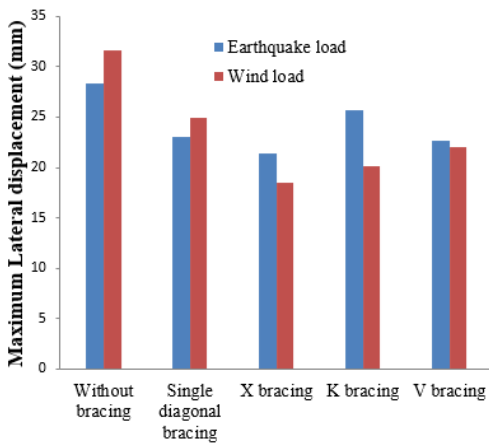


Figure 3.9(b) Lateral Displacements along Longitudinal direction

Fig. 3.9 Lateral Displacements

3.3.6

3.3.7 Column Axial Forces and Bending Moments

The variation of maximum axial forces (Compressive) and bending moments for columns due to the combined effect of static and lateral loading has been shown in the following Fig. 3.10.

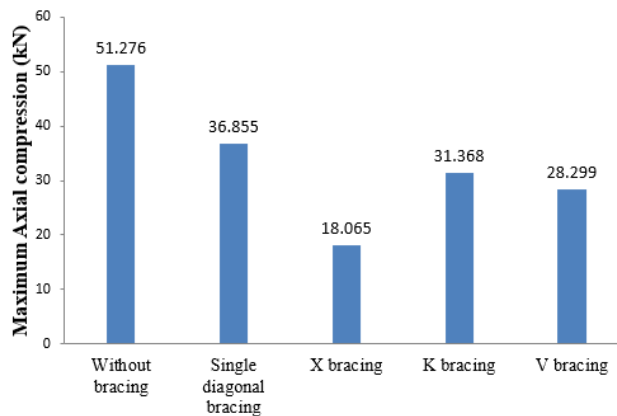


Fig. 3.10(a) Column Axial forces along Transverse direction

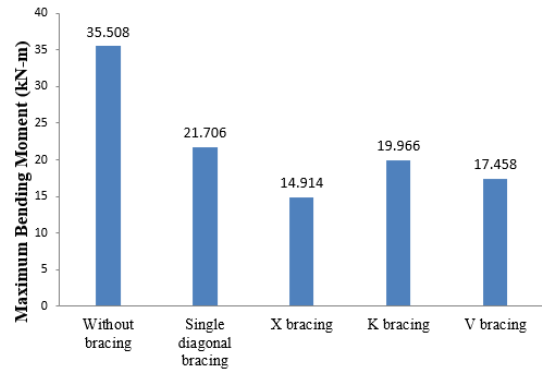


Fig. 3.10(b) Column bending moment along Transverse direction

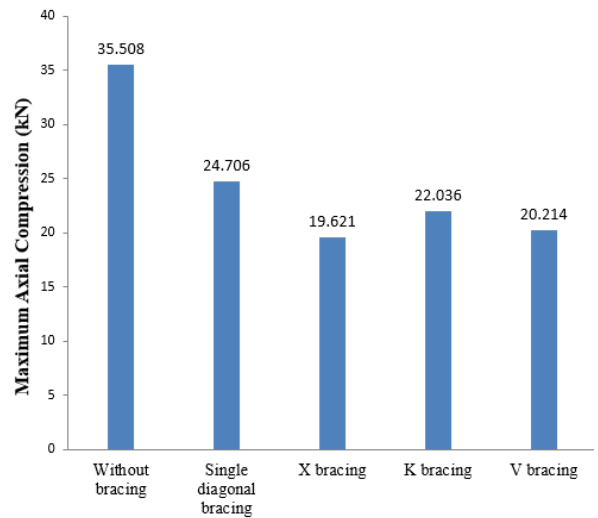


Fig. 3.10(c) Column Axial forces along Longitudinal direction

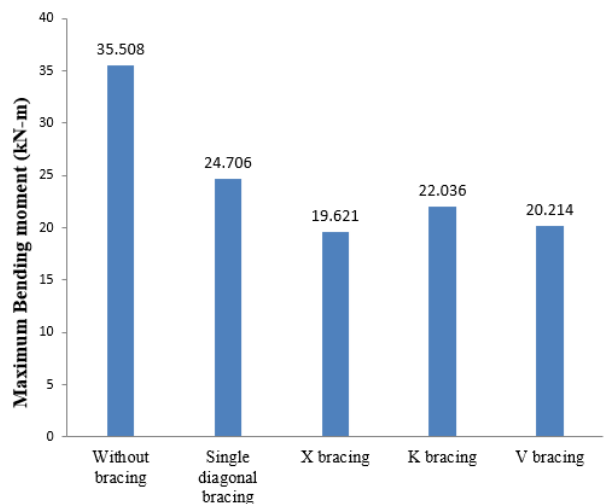


Fig. 3.10(d) Column bending moment along Longitudinal direction

Fig. 3.10 Column Axial Forces and Bending Moments in Longitudinal and Transverse

3.4 STABILITY ANALYSIS RESULTS

Both pinned base and fixed base support condition models were developed for analysis. The first model was analyzed with a pinned base column. The member sizes were chosen without regard to serviceability and picked only to satisfy the load demand. Direct analysis method was applied to the model and the results compiled. From, comparison of the linear elastic analysis with direct analysis method, the maximum ratio Δ_2/Δ_1 is 1.21. Because the ratio Δ_2/Δ_1 is less than 1.7 (reduced stiffness is used to calculate drift), notional load need only be applied in the gravity only load combinations as per AISC 360-10. For the representative pinned base model, stability analysis can amplify the deformation by up to 21% for this specific model. Deformation may not always be the focus of analysis and design but when checking serviceability limits; stability analysis can increase deformations significantly when compared to an elastic first order analysis.

Demand to capacity for members should also be used in stability analysis. The ratio of maximum demand to capacity for pinned base, for column, 0.665 (Linear and elastic analysis) and 0.76 (Direct analysis method) and for beam, 0.834 (Linear and elastic analysis) and 0.943 (Direct analysis method). The ratio of maximum demand to capacity for Fixed base, for column, 0.864 (Linear and elastic analysis) and 0.896 (Direct analysis method) and for beam, 0.835 (Linear and elastic analysis) and 0.868 (Direct analysis method).

The linear elastic analysis was included as a benchmark for comparison. The linear elastic analysis can be

seen to underestimate the demand to capacity ratios of members, sometimes significantly. When comparing the fixed base ratio Δ_2/Δ_1 for the direct analysis method, it can be seen that the maximum value is 1.07. While this is slightly less than for the pinned base support condition model, it still shows the significance of stability analysis in design.

When comparing the two support condition models, several observations can be made. The representative fixed base model tends to have slightly lower second order effects compared to the pinned based model. The fixed base model also tends to have lower deformations even when smaller member sizes are used. When demand to capacity is the only consideration in design, the deformations can easily become relatively significant and exceed standard serviceability limits especially in the case of pinned base support conditions.

3.5 STUDIES ON TUBULAR PIPE RACKS

Based on the linear elastic analysis carried out on pipe racks having conventional and tubular sections it is found that irrespective of the type of sections used X bracings provided along the transverse direction of pipe rack gives the optimal results. Hence as a further extension of the studies carried out the behaviour of pipe racks fabricated of Square, Rectangular and Circular sections with X-bracing configurations has been analyzed.

3.5.1 Dimensions of the Tubular sections

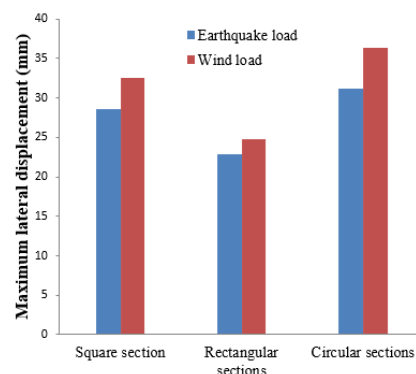
The dimension of the tubular sections used is shown in Table 4.14.

Table 4.14 Properties of Tubular sections

Members	Tubular sections		
	Square sections	Rectangular sections	Circular sections
Beams	45x45x2.6, 72x72x3.2 110x110x4.5, 120x120x4.5	50x25x2.9, 96x48x4.8 140x80x4.5, 145x82x5.4	O.D.127, O.D.139, O.D.42, O.D.88, t=5
Columns	110x110x4.5, 48x48x2.9, 150x150x5.8, 80x80x3.2	122x61x4.5, 140x80x5.4, 80x40x3.2, 65x32x3.7	O.D.140, O.D.127, O.D.76, O.D.114, t=5
Bracings	89x89x3.6	127x50x3.6	O.D.87 t=3

3.5.2 Lateral storey displacements

Fig 3.11 show the lateral displacement obtained with different shapes of Tubular bracing provision and compared with permissible limit of lateral displacement which is H/500 as per IS 875(Part 3).



3.5.2.1 Fig 3.11 Lateral Storey displacements of different sections of Tubular sections

3.5.3 Column Axial Forces and Bending Moments

The variation of maximum axial forces (Compressive) and bending moments for columns due to the combined effect of static and lateral loading has been shown in the following Fig 3.12.

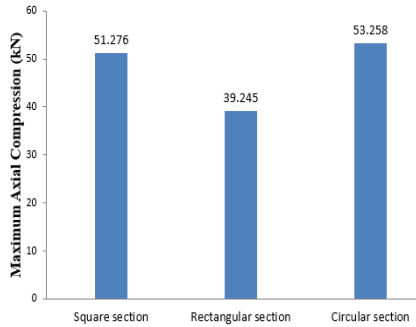


Fig 3.12(a) Maximum Axial compression of different sections of Tubular sections

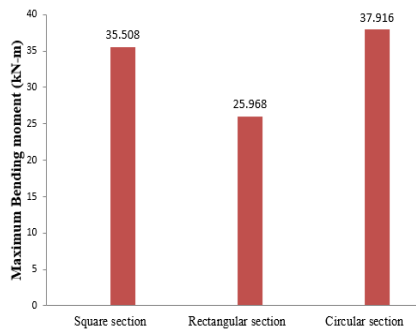


Fig 3.12(b) Maximum Bending moment of different sections of Tubular sections

Fig 3.12 Maximum Axial compression and Bending Moment of different sections of Tubular sections

3.6 COST BENEFIT STUDY

With the use of analysis results design is carried out for required load carrying capacity. Optimum sections are assigned to beam, column and bracing members. Comparison is made for self weight and cost of various elements of steel pipe rack structure. These results show that considerable amount of saving is achieved using Tubular sections. Results are presented in Fig3.13. Study reveals that considerable saving in cost can be achieved by using tubular sections.

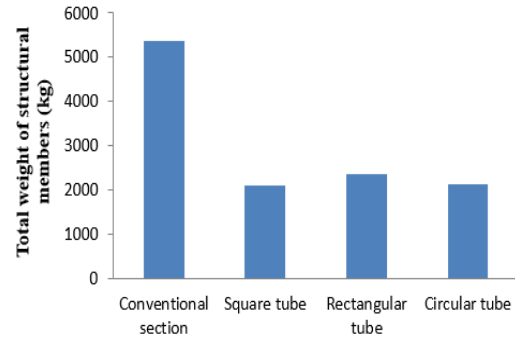


Fig 3.13(a) Variations of design weight for Steel pipe rack-Pinned base

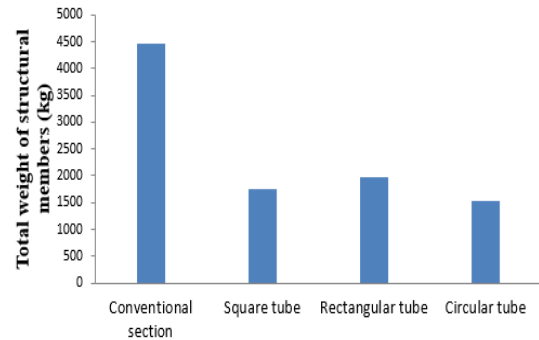


Fig 3.13(b) Variations of design weight for Steel pipe rack-Fixed base

Fig 3.13 Variations of design weight for Steel pipe rack

4 CONCLUSIONS

A detailed analysis has been carried out on continuous pipe rack systems. Pipe racks using conventional sections and tubular sections have been compared. In both conventional and tubular systems, the influence of different types of bracings viz., Single diagonal bracing, X bracing, V bracing, K bracing has been compared. Based on the elaborate analysis carried out the following conclusions has been arrived.

1. Linear elastic analysis is not sufficient especially when designing moment frames as the demand to capacity ratio could be underestimated by approximately 10-20% for the worst case when conducting a linear elastic analysis.
2. Varying the stiffness or geometry could easily produce greater errors in analysis if stability analysis is not considered.
3. Total saving of almost 50% to 60 % in cost can be achieved by using Tubular sections.
4. Due to connection difficulties of circular tube sections, Rectangular or Square tube sections can be adopted.
5. Members having larger unsupported lengths can be assigned tubular sections which will enhance the overall economy.

6. Based on the linear elastic analysis carried out on pipe racks having conventional and tubular sections it is found that irrespective of the type of sections used X bracings provided along the transverse direction of pipe rack gives the optimal results.

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