STABILITY ANALYSIS OF STEEL STRUCTURES
CODAL PROVISIONS - IS:800-2007 AND AISC:360-05

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STABILITY ANALYSIS OF STEEL STRUCTURES CODAL PROVISIONS - IS:800-2007 AND AISC:360-05

Major Project

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For the degree of

Master of Technology in Civil Engineering

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By

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Declaration

This is to certify that

a. The thesis comprises my original work towards the Degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) at Nirma University and has not been submitted elsewhere for a degree.

b. Due acknowledgment has been made in the text to all other material used.

Mehul V. Pethani

Certificate

This is to certify that the Major Project entitled "Stability Analysis of Steel Structures - Codal Provisions - IS:800 2007 and AISC 360-05" submitted by Mehul V. Pethani (09MCL012), towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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Abstract

Stability is the fundamental safety criterion for steel structure. Structural or geometrical instability occurs due to compressive load and is usually known as buckling. The buckling load is the maximum load at which the compression member becomes unstable.

There are many analysis methods of varying degree of detail and preciseness available at structural engineer's disposal. Higher end analysis types are meant to consider more realistic (near to real life conditions) aspects and less 'ideal' assumptions; but obviously they involve more time consuming and complex procedure. The decision to select an appropriate analysis type is always of paramount importance and therefore has to be made judiciously. Stability requirements and related codal provisions provide here very useful guideline for selection of proper analysis method. New codal provisions also make the design aspect 'simpler' by eliminating use of certain amplification factors used by earlier code (e.g. Effective length factor K) when refined analysis method is employed.

Nonlinearity e.g. geometrical imperfections, material imperfections, residual stresses etc. affects the structural stability significantly and is therefore needed to be considered either at analysis stage or at design stage. To consider nonlinearity at design stage, codes have introduced factor called "effective length factor" - K, in addition to the moment multiplication factor used during member strength check. The effective length factor is just a mathematical adjustment to enable application of Euler theory to say frames and to consider the above nonlinearity effects. Evaluation of 'K' factor has always been an intricate task for Structural engineers. Hence, it's a relieving aspect of new advancement of Stability Design highlighted in this work that by explicitly considering the above listed aspects the 'K' factor can be set as 1. The interesting history behind the birth of the 'K- factor' up to it's cessation has be traced

in this work.

In the present study, two eminent steel design codes IS:800-2007 and AISC:360-2005 are studied with special focus on provisions meant for ensuring structural stability and related analysis requirements. It is interesting to note that both the codes have come up with new but similar stability provisions superseding respective earlier edition of the codes. The new provisions are based on recent research work in this field and they provide good insight into structural behaviour and it's failure pattern under buckling case as well as various practical factors affecting the buckling phenomenon.

To consider nonlinear effect at analysis time and design using K=1, AISC:360-2005 has presented a new versatile method called "Direct Analysis Method". Other two methods second order analysis and first order analysis have a limited use whereas direct analysis is applicable to all types of structure. The direct Analysis method is of particular relevance for the Structural Engineers as it is described as 'The Future of Stability Analysis' by AISC specification committee chairman Mr. Shankar Nair and it is the main/mandatory method of the contemporary 2010 edition of AISC:360 published in 2011.

The New edition of Indian standard IS:800-2007 presents various methods of analysis of steel structure with regard to stability. There are three approaches permitted: (1) First order analysis and moment amplification during design, this has limited application though, (2) Second order elastic analysis, (3) Advanced structural analysis. A notable observation on IS:800-2007 code is the need of further explanation for implementation of each of these methods.

Piperack structure is an important and the most common structure in the industrial plants. Hence, a typical piperack structure is selected to study and demonstrate impact of new provisions for stability analysis. STAAD Pro being the most popular software package of field is used for analysis and design.

The piperack structure selected as Case Study problem is solved with various methods prescribed by Codes AISC:360-2005 and IS:800-2007. The results are compared to illustrate the variation. For academic purpose the case-study is also solved as per IS:800-1984 provisions and compared with results from 2007 edition.

The present work is aimed at exploring a relatively complex phenomena of Structural Engineering : 'Stability analysis of steel structure' with special focus on relevant codal provisions. With example of case study, a 'real-life' structure has been solved to demonstrate the application of all the background theories explained in the body of work. In practical perspective, this work deciphers the latest codal provisions related to structural stability and further validate and justify it's application by exemplifying through Case-Studies.

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-Mehul V. Pethani

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Abbreviation Notation and Nomenclature

A_g .... Gross area of the cross section

AISC .... American Institute of steel construction

ASCE .... American Society of Civil Engineers

ASD .... Allowable Stress Design

C .... Connection factor

C_{mLT} .... Equivalent uniform moment factor for lateral torsional buckling

EID .... Equivalent Initial Deflections

F_e .... Elastic critical buckling stress

F_{cr} .... Flexural buckling stress

L_{eff} .... effective length

L_i .... Actual height of column ‘i’

LRFD .... Load and Resistance Factor Design Specification

M_{dy}, M_{dz} .... Design strength under corresponding moment acting alone

M_{lt} .... First-order moment caused by lateral translation of the frame only

M_{nt} .... First-order moment, assuming there is no lateral translation of the frame

M_r .... Required second-order flexural strength

M_y, M_z ..... Maximum factored applied bending moments about y and z-axis of the member respectively

η .... Member stiffness index

P_{cr} .... Buckling Load of Column

P_d .... Design strength in compression due to yielding

P_{dy} ..... Design strength under axial tension or compression as governed by buckling about y axis

P_{dz} ..... Design strength under axial tension or compression as governed by buckling about z axis

P_{nt} ..... First-order axial force, assuming there is no lateral translation of the frame

$\sum P_{nt}$ .... Total vertical load supported by the story, including gravity column loads

P_{lt} .... First-order axial force caused by lateral translation of the frame only

P_{e1} .... Elastic critical buckling resistance of the member in the plane of bending, calculated based on the assumption of zero sidesway

P_r ..... Required axial compressive strength under LRFD or ASD load combinations

P_y .... Member yield strength

N_i .... Additional lateral load

Y_i .... The total gravity load on level i

SSRC .... Structural Stability Research Council

Δ_{oh} .... Inter-story deflection

$\sum_H$ .... Story shear produced by the lateral forces used to compute

Δ_H .... First-order interstory drift due to lateral forces

γ_{mo} .... Partial factor of safety in yielding

$\sigma_{bcx,cal}$ .... Calculated bending compressive stress due to the bending moment about major axis

σ_{bcx} ... Permissible bending compressive stress about major axis taking into account lateral instability

$\sigma_{bcy,cal}$ .... Calculated bending compressive stress due to the bending moment about minor axis

σ_{bcx} ... Permissible bending compressive stress about minor axis taking into account lateral instability

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Chapter 1

Introduction

1.1 General

Stability is the fundamental safety criterion for steel structures during their construction period and operation life. Although research on the stability of structures can be traced back to 250 years ago when Euler published his famous Euler equation on the elastic stability of bars in 1744, adequate solutions are still not available for many types of structures while subjected to certain load conditions.

This chapter explains the basic fundamental of stability design, different methods of stability analysis and birth of K factor in stability of structure.

A structure is meant to withstand or resist loads with a small and definite deformation. In structural analysis problems, the aim is determine a configuration of load resisting system, which satisfies the condition of equilibrium, compatibility and force displacement relations of the material. For structure to be satisfactory, it is necessary to examine whether the equilibrium configuration so determined is stable.

In practical sense, an equilibrium state of a structure or a system is said to be in a stable condition, if a disturbance due to accidental forces, shocks, vibrations, eccentricities, imperfections, inhomogeneities or irregularities do not cause the system to depart excessively from that state. The usual test is to impart a small disturbance to the existing state of the system, if the system returns back to its original undisturbed state when the cause of disturbance is removed, the system is said to be stable[2].

There are two types of failure associated with structure namely material failure and form or configuration failure. In the former, the stresses exceed the permissible values which may result in the formation of crakes. In the later case, even though the stresses are within permissible range, the structure is unable to maintain its designed configuration under the external disturbance. The loss of stability due to tensile loads falls in the broad category of material instability, whereas the stability loss under compressive load is usually termed structural or geometrical instability commonly known as buckling, see Figure 1.1. A buckling failure is potentially very dangerous and may

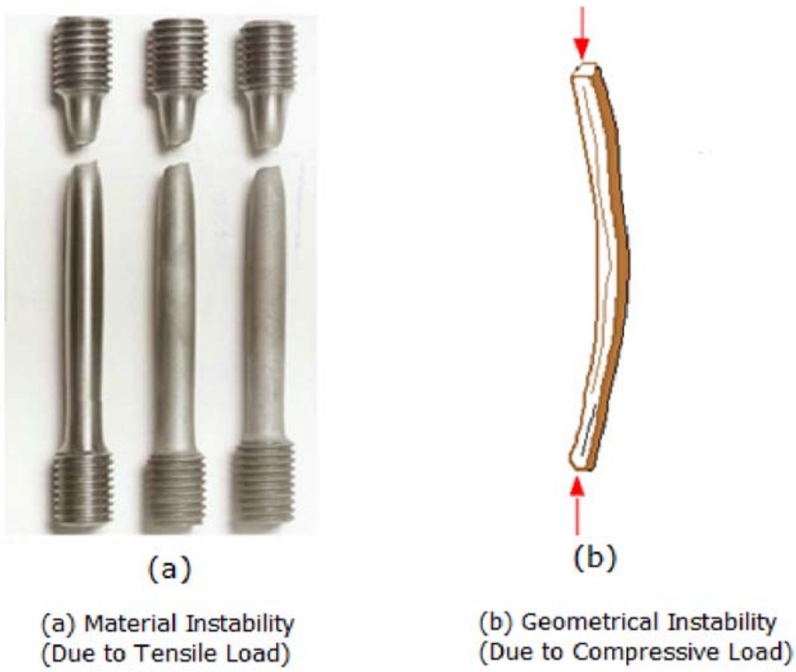


Figure 1.1: Types of Structural Instability

trigger the collapse of many type of engineering structures. It may take the form of instability of the structure as a whole or localized buckling of an individual member or a part there of, which may or may not precipitate the failure of the entire structure.

It is to be emphasized that load at which instability occurs depends upon the stiffness of the structure or portion there of, rather than on the strength of material[2].

Instability is a condition wherein a compression member loses the ability to resist increasing loads and exhibits instead a decrease in load - carrying capacity. In other word, instability occurs at the maximum point on the load deflection curve[2].

To determine the failure load of an actual member it is necessary to take initial imperfection into account and to consider the entire nonlinear load deflection curve of the member. To consider actual nonlinear behaviour various analysis methods have been proposed.

1.2 Different Methods of Frame Analysis

Numerous approaches have been proposed to consider nonlinear behaviour of the structure. Out of the different methods Linear elastic analysis is the most common, although the least absolute while the second-order analysis is the most comprehensive and most complex.
Numbers of methods proposed for analysis as follows:

- First order elastic analysis
- Second order elastic analysis
- First order elastic plastic analysis
- Second order inelastic analysis

In first order analysis, the deformations are determined and then used in turn to calculate the forces. Once the forces are calculated, the analysis is complete. first order analysis method assume that the deformations are small and will not produce any additional forces. i.e. load-deflection curve is linear. If deflections are large enough, then the equilibrium equations need to be applied to deformed geometry of the structure to consider second order effect.

The moments from first order analysis are lower than the moments from second order analysis. The second order analysis is also known as geometric nonlinear analysis. The second order moments are produced due to the member curvature between the supports. This effect is known as P-delta effect see Figure 1.2.

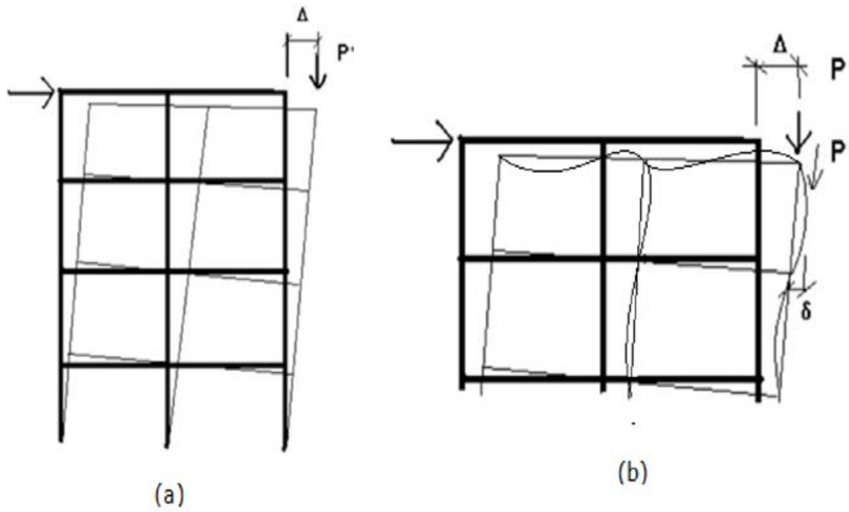


Figure 1.2: P-delta effects (a) P-Δ: a structure effect, (b) P-δ: a member effect

Second-order analysis when accounting for P-Delta combines two effects to reach a solution:

a. Large displacement theory - the resulting forces and moments take full account of the effects due to the deformed shape of both the structure and its members.

b. "Stress stiffening" - the effect of element axial loads on structure stiffness. Tensile loads straighten the geometry of an element thereby stiffening it. Compressive loads accentuate deformation thereby reducing the stiffness of the element.

Application of these all advance analysis methods permits a comprehensive assessment of the actual failure modes and ultimate strength of structural steel system in stability of structure and practical design situation. For detail design concept related to all above methods refer chapter 3 “Methods of analysis”.

1.3 Fundamental Concepts in Stability Design

Before exploring more about the codal provisions, the fundamental concepts in stability need to be looked into greater depth. In this section Euler's buckling, Stability of sway and non sway frames, side sway buckling are presented.

1.3.1 Stability of Non-Sway Frames

The axial strength of the column is calculated by buckling load of the column. Buckling load is defined as the critical load at which an ideal column become unstable due to it's slenderness. This phenomenon is called as buckling. The buckling load is the direct axial load with no transverse load to cause bending.

This buckling was first studied by Euler and it is called as Euler's buckling or elastic buckling load. This is applicable to “non sway” long columns. The equation for buckling load is given by,

for pin ended column.

$$P = \frac{\pi^2 EI}{L_{eff}^2} \tag{1.1}$$

L_{eff} = effective length of column.

For different end conditions effective length L_{eff} is different. Typical example is, when the ends of the column are fixed then $L_{eff} = 0.5L$ or $L_{eff} = KL$ where $K = 0.5$, K is effective length factor. This introduced “the Birth of the concept K” by Euler for non sway frames. Euler’s buckling is mainly applicable for non sway frames. Max value of K is 1.0 for pin ended column. Hence K is the mathematical adjustment to express all the columns in terms of pin ended column.

As shown in Figure 1.3 P_e is the Euler’s buckling load before which the column

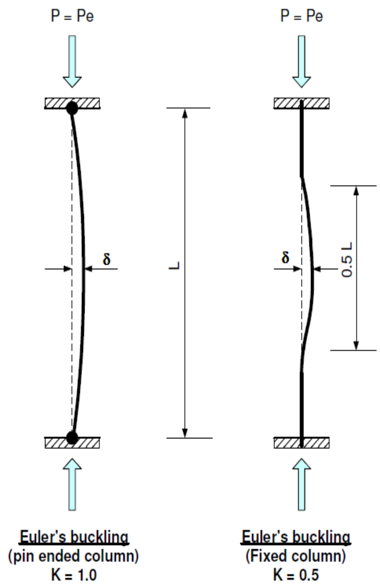


Figure 1.3: Concept of K for Non-sway Frames

is in neutral equilibrium. At P_e it changes it’s equilibrium condition from neutral to unstable equilibrium and undergoes large deflection. This is known as bifurcation since; the neutral equilibrium condition bifurcates and goes in unstable condition.

The following Figure 1.4 shows the bifurcation buckling and graph shows the meaning of bifurcation point. The buckling load is also known as bifurcation load.

This buckling phenomenon is elastic and if the load is removed then the column

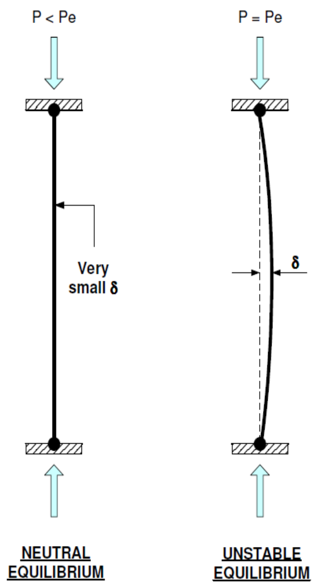


Figure 1.4: Bifurcation Buckling

comes back to it's original shape. This is the ideal column behavior and applicable for flat yielding steel. The hot rolled sections, due to residual stresses yield gradually.

The yielding of some portion of the section will start before yield point. Hence the deflection corresponding to the load ($< P_e$) is more than the flat yielding steel. The Figure 1.5 shows the actual column behavior having more deflection.

1.3.2 Stability of Sway Frames

As shown in the Figure 1.6 non sway frames and sway frames buckle in a non sway mode and sway mode respectively. A sway frame can not undergo non sway buckling mode. Euler's theory can be extended to sway frames. Consider an a frame with both end hinged and infinite beam stiffness. The frame undergoes buckling in a side sway mode as shown in the Figure 1.6.

To apply Euler's theory, for non sway frame as shown in Figure 1.7 the L_{eff} be-

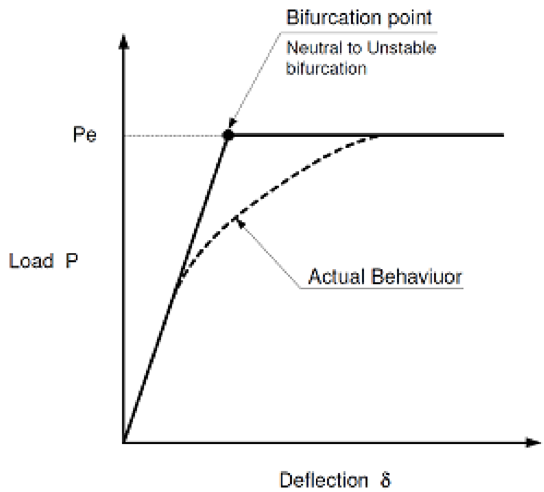


Figure 1.5: Ideal Column Behavior - Bifurcation Buckling

comes 2L. It means that K becomes 2.0. Hence load carrying capacity becomes $1/4^{th}$ of pin ended column having length L and this simplifies the stability problems of sway frames. It can be easily noted that the Birth of the concept K was made for application of Euler's theory to sway frames.

1.4 Column Buckling Phenomena

It is already narrated above that, all the sway frames will undergo sway buckling.

1.4.1 Ideal Column

The ideal column can be defined as a perfectly straight column with no geometrical and material imperfections. Ieal column or ideal frame will undergo a non sway buckling failure due to slenderness. So this behavior is matching with the non sway behavior. The sway is not taking place since there is no apparent load applied in transverse direction and there are no imperfections in the column. K value can be taken as 1.0. Hence in nutshell K will be always less than or equal to 1.0 for ideal columns.

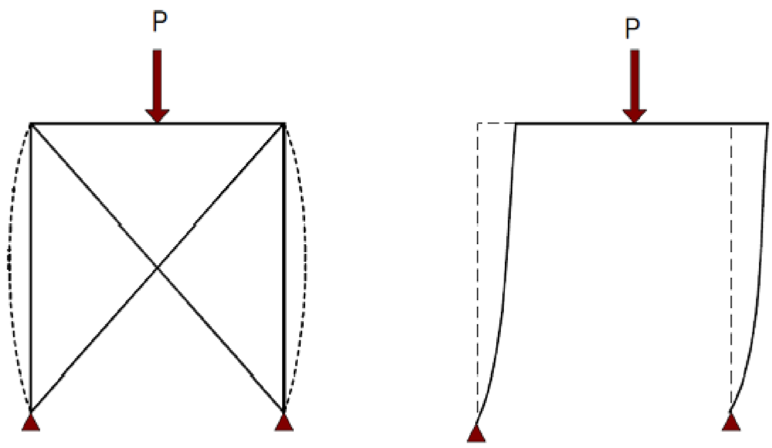


Figure 1.6: Sway And Non Sway Buckling

1.4.2 Practical Column

The structures which we design do not have the ideal conditions. All the columns have geometrical and material imperfections. So, this needs to consider in design or analysis stage.

Geometrical Imperfections

The geometrical imperfection can include out of straightness, out of plumbness, fabrication and erection tolerances etc. The column can not be erected perfectly straight and will have some out of plumbness. AISC allows the out of plumbness of column as 1 in 500. Refer the Figure 1.8 showing out of plumbness.

For proper erection of the steel members, we use fabrication and erection tolerances. So the length of column or beam can be short or more by few millimeter. This also creates eccentricity in the overall frame structure. Due to these and similar problems, the inherent eccentricity is generated and the practical column will buckle in the sway mode.

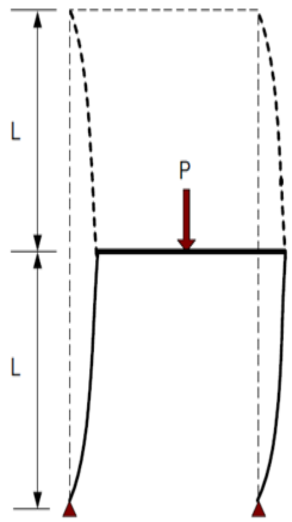


Figure 1.7: Simplification of Sway Buckling Problem

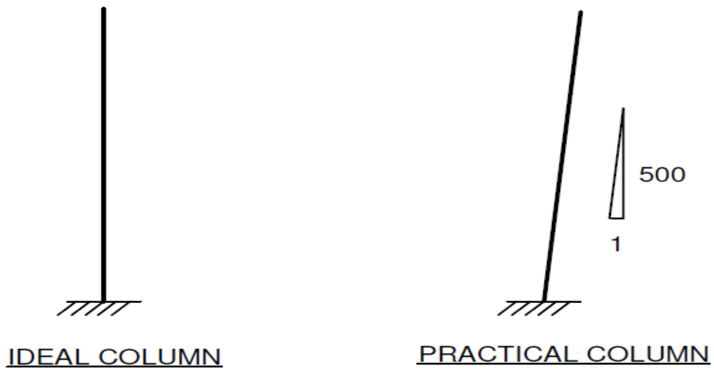


Figure 1.8: Out of Plumbness

Material Imperfections

The steel sections which are either Hot rolled sections or Welded sections. In both cases, stresses are generated due to hot rolling or welding. Due to these stresses, stiffness of the member reduces. As shown in Figure 1.9 ideal column will undergo yielding at a single point where as Practical columns (hot rolled or fabricated) will undergo gradual yielding.

For ideal column (flat yielding steel), there is a single yield point before which

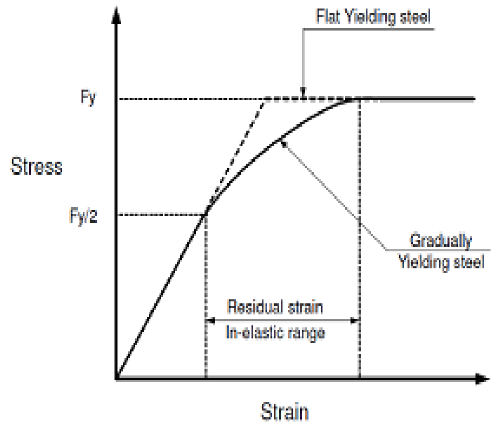


Figure 1.9: Gradually Yielding Steel

stress is proportional to strain.

For practical columns, the gradual yielding will start at $F_y/2$ as shown and finally fully yield at F_y. The strain will be developed in the columns between $F_y/2$ and F_y is called as residual strain. This is also called as inelastic strain.

Residual Stresses

There is non uniform cooling of flanges and webs because in hot rolled sections, the exposed surface area of flanges and central portion of web is more than the joints as shown in Figure 1.10. Because of this reason, some stresses are developed internally called as residual stresses. Tips of the flanges and middle portion of the web are in compression while the joints are in tension.

f_{rc} = Residual compressive stress.
f_{rt} = Residual tensile stress.
As shown in Figure 1.10 due to these initial stresses, tip of flanges and central portion of the web will start yielding before the joint portions. The amount of residual stress is about 33% of yield stress.

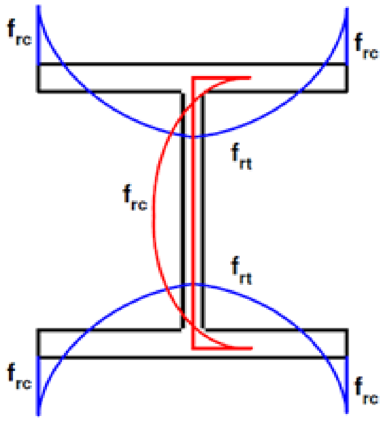


Figure 1.10: Residual Stresses

This is the effect of residual stress on the strength of the column. This behavior of column to undergo premature buckling or lateral torsional bucking under the load less than it's actual elastic buckling load is known as in-elastic failure. Columns which are not very long and not very short will normally undergo inelastic buckling failure(i.e. all practical columns). To take into account this effect the effective cross section and the moment of inertia are reduced thereby reducing bending stiffness (EI_{eff}) and buckling capacity of column.

This is valid for beam for which the compression flange will buckle in-elastically thus undergoing in-elastic lateral torsional buckling. These imperfections are important and will reduce the capacity of the column substantially. The effect of residual stresses on the strength of the column can be taken into account either at analysis stage or at the design stage.

For reducing K = 1.0, The practical imperfections have to be modeled in analysis time.

1.5 Objectives of Study

The main objective of this study is to understand the impact of stability analysis on design of plant structure by using old and new provisions of IS:800 and AISC:360 standards.

The key objectives of study are as follows:

- To study the different analysis type and it's impact on design of plant steel structure.
- To study the effective length factor K and its significance in the stability analysis.
- To study provisions related to stability analysis of Indian (IS:800) and international(AISC:360) standards new and previous version.
- To carry out stability analysis of design of steel piperack by considering IS:800 and AISC:360 provisions and compare its impact on design result.

1.6 Organization of major project

The content of major project is divided into different chapters as follows:

Chapter 1, presents the introduction and overview of the major project work. The various methods of structural analysis and its impact on stability design is discussed. The fundamental concept of stability for sway and non-sway frame along with column buckling phenomena is also described.

Literature review is discussed in **Chapter 2**. In this chapter brief literature review is presented pertaining to stability analysis of steel structures, various analysis methods of stability of structure and assessment of K factor formula.

Structural analysis methods are presented in **Chapter 3**. The introduction of each analysis methods and its significance is also discussed. To study different methods of analysis a example of 2-D frame, one bay three story frame is considered. Moment and deflection are calculated with different analysis methods.

Chapter 4 deals with the effective length factor K and importance of K factor in stability design. The procedure to find K factor by different methods such as AISC Alignment chart method, lemessurier's method, lui's method, IS:800-1984 and IS:800-2007 method is explained. A numerical example is illustrated to find the effective length factor K by all these methods.

Chapter 5 explains the stability analysis provision based on AISC:360-05(American institute of steel construction). Comparison of all three analysis method such as first order analysis method, second order analysis method and direct analysis method has been shown by solving one bay two storey frame structure.

Stability analysis provision as per new Indian standard IS:800-2007 is explained in **Chapter 6**. An example structure is analyzed and design according to IS:800-2007 as per section 4 and 5, as per Appendix B and IS:800-1984.

In **Chapter 7** a case study-I is presented. A piperack structure from a past project data is selected as case study, with its geometry and loading data taken as input. The structure is analysed and designed by different methods namely (1) First order analysis (2) Second order analysis (3) Direct analysis method prescribed by AISC 360-2005 for ensuring stability of steel structure. The results of different methods and their interpretation to show the impact of stability analysis criterion are presented.

In **Chapter 8** a case study-II is presented. Same piperack structure has been taken as for study as mentioned in Chapter 7. The structure is analysed and designed by different methods namely (1) First order moment amplification analysis as per IS:800-2007 (2) Advance analysis IS:800-2007 (3) As per IS:800-1984 . The result of different methods and their interpretation to show the impact of stability analysis criterion are presented.

Chapter 9 summarizes the work done in the major project. Chapter includes summary of work done, various conclusions obtained from the study and future scope of work.

Chapter 2

Literature Survey

2.1 General

Literature survey is carried out to review various criteria which are to be considered for the stability analysis and design of steel structure. This chapter explores study of various papers, books and journals to understand the basic concept of stability analysis.

2.2 Literature Review

Various literatures related to stability analysis of steel structure are studied and brief review is presented.

2.2.1 Books and Guidelines

Indian Standard (IS) 800-2007[3] published the "general construction in steel" a design standard for structural steel. Standard specify the different methods such as advance analysis methods, second order analysis and frame instability analysis. Also specify the calculation of effective length of column in frame and effective length for steeped column and double column.

American Institute of Steel Construction (AISC)[1] specification contains significant changes to method available for stability analysis and design of steel structures. The AISC specification includes three methods of analysis: Direct Analysis Method, the Effective Length Method, and the First Order Analysis Method and application in to design.

The book of **Asvinikumar[4]** discusses the fundamental concepts stability of structure. Method of analysis of large deflection and effect of small imperfection on Stability are included Author also discussed the dynamic stability of structures.

The book for stability analysis and design of structure by **Gambhir[2]** focused on basic principles of stability analysis. Different analysis method for stability of structure is discussed. Literature include Stability analysis of beam column, buckling analysis of axially loaded member, stability analysis of steel frame with illustration are explained. The American national standard, Australian code AS:1250-1981, British code BS: 5940-1985(part-1) and Indian code IS:800-1984 has been compared for the provisions related to stability consideration and design illustrations also has been demonstrate.

Indian Standard:800-1984[5] published the "general construction in steel" a design standard for structural steel, Second Revision. To calculate the effective length IS:800-1984, in appendix C based on wood's curve has been presented based on the ratio of l/L effective length l to unsupported length L. It is also recommended that the effective length ratio l/L may not be taken to be less than 1.2.

Subramaniyan[6] the book on Design of Steel structures based on the limit state method of design as per the I:S800-2007. Book provide wealth of information regarding concepts of different methods of analysis, assumption with each type of analysis and effect on analysis in design of steel structure with illustration. Author also describe the different type of method to calculate the effective length methods and comparison with IS:800-2007 method.

Abhijit[**7**] presented concepts on Stability of sway and non sway frames. Litructure covers the study of K and design of beam-column, right from AISC-ASD edition upto unified steel code 2005 which includes direct analysis method. Author presented the fundamental concepts in stability design, second order analysis and birth of K factor. Also covered canadian code provisions for effective length and second order analysis in brief and about notional load approach.

2.2.2 Analysis methods

Various methods of analysis e.g. First-Order elastic analysis, Elastic buckling analysis, Second-Order elastic analysis, First-Order plastic-mechanism analysis, First-Order elastic-plastic analysis and Second-Order inelastic analysis were briefly reviewed and presented by **Geschwinder**[**8**]. Difference between first order elastic analysis, second order elastic analysis and elastic buckling analysis were carried out using GTSTRUDL(1999) including axial, flexural, shear deformation and difference of result showed that lateral displacement increases progressively as the magnitude of the load increased. In addition to that impact of two different second order effects on a single column was studied by author.

First order elastic buckling analysis for one storey four bay frame with leaning columns was performed using GTSTRUDL and effective length factor K was evaluated using four different approaches. From analysis result it was observed that leaning columns have a significant impact on the stability of structure.

Nair[**9**] presented a model specification for stability analysis by direct method. The purpose or physical significance of each of the important steps in the direct analysis method was outlined in the paper showing the correlation of these steps to the basic requirements for design of structures for stability. Author suggest that the two method effective length method and first order method has limited applicability were as the direct analysis method is applicable to all structure.

In this paper **Masarira[10]** explained the various commonly used beam to-column connections with various stiffening arrangements they were analyzed in order to determine their effect on the stability behaviour of the whole structure. Author compared the result between the critical loads obtained from the finite element method, and those computed from the equivalent-member method. This study has contributed towards a more accurate evaluation of the structural stability of frames.

Oda[11] presented a stability design method for steel frames based on second order elastic analysis. The introduction of equivalent initial deflections (EID) has been calculated assuming a pin-ended column subjected to a concentric compressive force and equal moments at both ends is considered and proposed a formula to calculate the magnitude of EID.

Wang[12] explained the variabilities of loads and locations that need to be accounted for when assessing the stability of structures. Numerical example on 2-bay 2-storey steel frames with different connections was carried out for five different cases with variables:
Column base connections
Beam-to-column connections
a. Interior column.
b. Exterior column.

Bendapudi[13] discussed the effect of ambient temperature changes, expansion joint requirement, design loads, structural stability and detail for stability for pipe support structures. Author explained that frame instability occur due to initial eccentricities, fabrication and erection tolerance, dead load and the elastic deformations. Primary bracing system such as transverse braces, longitudinal brace and plan bracing arrangements to achieve frame stability was also discussed by author. Author also recommended that expansion joints are not required in any piperack of less than 150 meter long. All interior hanger or trapeze type pipe support should be braced in both direction for seismic loads.

Justion[14] discussed the effect of geometric imperfection with an emphasis on frame non verticality or out of plumbness. Main objective of the study was to illustrate how initial imperfection on the strength of members and framing system, magnitude and distribution of internal member forces and assessment of frame stability. Parametric study of 25 frames was analyzed with or without imperfection with respect to number of parameters, including slenderness ratios, leaning load levels, gravity-to-lateral load ratios, and lateral frame stiffness, as measured by a second-order to first-order drift ratio. It is observed that the AISC provision Appendix-7, in which the effects of imperfections may be neglected in lieu of higher lateral loads when $B_2 < 1.5$, is shown to produce a maximum unconservative error of 8%.This error occurred in a highly stability-critical portal frame laterally supported by a weak axis column only.

2.3 Effective length factor K

Various methods for K determination was reviewed, explained and summarized by **Chen[15]** Four different approaches i.e.(i) alignment chart (ii) Lemeessurier's Formula (iii)Lui's formula and (iv) system buckling method were considered to compute K formulas of columns. K factor for single storey single bay unbraced frame with uneven distribution of geometry, one single bay three storey frame, three two bay three storey frame and one 3-bay 10-storey frame was investigated by all the four methods.

Results of K factor evaluation showed that all methods except the alignment chart were found to predict nearly identical values. Authors also concluded that Lui's formula was the most simple, effective and appropriate for general use.

Lui[**16**] presented a simple straightforward approach for determining the effective length factors for column main objective of the study was to demonstrate that the proposed method for K factor determination was applicable for all frames. K factor for column was calculated for three different type of frame i.e. (i) simple portal frame with leaner columns and (iii) 2-storey 2-bay frame to the validity of proposed approach. K factors computed using proposed formula provide accurate estimates of K factor by incorporating both member instability effects explicitly without using any special chart.

Farshi[**17**] proposed iterative procedure based on AISC code for allowable stress design provision (ASD) to determine overall frame stability with true safely factor. The 2 dimensional 3 bay 5 storey steel frame, with and without lateral bracing was analyzed to illustrate proposed method. A unique buckling factor for the whole structure was evaluated to consider length factors K for columns which was computed using proposed method and compared with those obtained by different authors. From that result, it was clearly observed the convergence was effectively achieved in reasonable number of iteration for all cases of unbraced frame and frames with various types of bracings.

2.4 Summary

In this chapter, review of relevant literature is carried out. The review of literature includes different types of analysis method, calculation of effective length factor K and impact of K factor in design of structure.

Chapter 3

Different Types of Analysis

3.1 General

In structural engineering practice analysis and design of frames is an integral activity. Numerous analysis methodologies are available for solving complex structural engineering problem. Many commercial software packages are available, for application of different complex analysis methods.

In this chapter, various methods for analysis are explained with assumption and impact of different analysis types. An example is solved to illustrate main analysis types of interest for present work.

3.2 Types of Analysis

Linear elastic analysis is perhaps the most common, although the least complete. A second-order inelastic analysis, while perhaps the most comprehensive, it is most complex as well. And there are many approaches between these. Whichever analysis method is chosen, the design approach must be compatible.

The various type of analysis that can be used are given below.

Analysis Methods:

- First order elastic analysis.
- Second order elastic analysis.
- First order elastic plastic analysis.
- Second order inelastic analysis.

  a. Plastic zone method.

  b. Elastic plastic hinge method.

  c. Refined plastic analysis.

  d. National load plastic hinge method.

  e. Quasi-plastic hinge method.

3.2.1 First-Order Elastic Analysis

The first and most common approach to structural analysis is the first-order elastic analysis, which is also called simply elastic analysis. In this case, deformations are assumed to be small so that the equations of equilibrium may be written with reference to the undeformed configuration of the structure. Additionally, superposition is valid and any inelastic behavior of the material is ignored. Thus, the resulting load-displacement curve shown in Figure 3.1 is linear. This is the approach, which used in the development of the common analysis tools of the profession, such as slope deflection, moment distribution and the stiffness method that is found in most commercial computer software.

First order elastic analysis is sufficient for normal framed structure. various assumptions of first order elastic analysis are as follows.

a. The materials behave linearly and all yielding effects can be ignored.

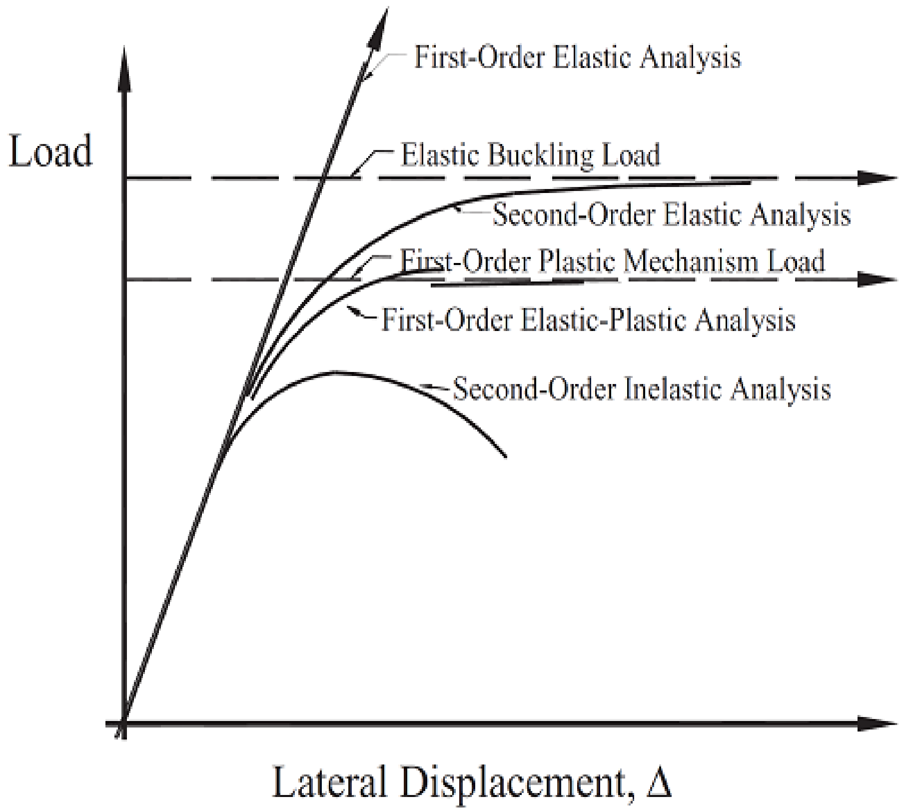


Figure 3.1: Load-displacements Relationship

b. The members behave linearly, and the member instability effects such as caused by axial compression, which reduce the member flexure stiffness, can be ignored.

c. The frame behaves linearly.

Several manual methods are available for the first-order elastic analysis such as the slope deflection method and moment distribution method. The advent of computer and the development of matrix method of analysis resulted in the development of numerous software packages like STAAD Pro, ETAB and SAP2000.

(a) Elastic Buckling Load

Elastic buckling analysis is used in the determination of a single critical buckling load for a system. The critical buckling load determined through an eigenvalue solution or through a number of iterative schemes is based on equilibrium equations.

Linear buckling load is calculated by linear buckling bifurcation analysis. The buckling load are obtained from the solution of elastic frames subjected to idealized loads that do not produce direct bending of the structure.

This analysis can provide the critical buckling load of a single column and is the basis for the effective length factor. It can be seen form Figure 3.1 that the results of this analysis do not provide a load-displacement curve but rather the single value of load at which the structure buckles.

3.2.2 Second-Order Elastic Analysis

In second order analysis material is assumed to behave like linear elastic relationship. However, the equations of equilibrium are written with reference to the deformed configuration of the structure and the deflections corresponding to a given set of loads are determined.

In first order analysis, the unknown deformations can be obtained in simple and direct manner, whereas second-order analysis requires an iterative procedure to obtain the solutions. This is because the deformed geometry of the structure is not known during the formation of the equilibrium and kinematic relationship. Thus the analysis proceeds is a step by step incremental manner, using the deformed geometry of the structure obtained from a preceding cycle of calculation.

For most practical case, accurate second order design forces can be obtained by applying the loads in one or two increments, and only a few iteration are required to converge to an accurate solution. The iterative process used in non liner solution can take up a large amount of computer time and may diverge from the required result. Therefore, the objective of selecting analysis methods is to reduce time and preserve the stability solution.

Second order analysis considers both member curvature (P-δ) and sides–sway (P-Δ) stability effects so, it is referred to as a P-delta analysis. The influence of member curvature is included, it is said that the P-δ effects or member effects are included and when the sidesway effects are included it is said that the P-Δ effects, also referred to as the story sway or frame effects are included.

The second order elastic analysis can account for all the stability effects, it does not provide information on the actual inelastic strength of the structure.

The load-displacement history obtained through this analysis may approach the critical buckling load obtained from the eigenvalue solution as shown in Figure 3.1.

(a) First-Order Plastic-Mechanism Load

Assumption with first order plastic analysis is that as the load is increased on a structure, certain critical locations within the structure will reach their plastic capacity. When this happens, the particular location continues to resist that plastic moment but undergoes unrestrained deformation. These location are called plastic hinges.

Once a sufficient number of plastic hinges have formed so that the structure will collapse, it is said that a mechanism has formed and no additional load can be placed on the structure. Thus, a plastic-mechanism analysis can predict the collapse load of the structure. The limit of plastic mechanism analysis is shown in Figure 3.1.

3.2.3 First-Order Elastic-Plastic Analysis

If the determination of the collapse mechanism tracks the development of individual hinges, more information, such as deflections and member forces, is obtained from this analysis. It is clear that if zero length hinges are assumed and the geometry is maintained, the limit of the elastic-plastic analysis will be the mechanism analysis as seen in Figure 3.1.

3.2.4 Second-Order Inelastic Analysis

Inelastic analysis refers to any method of analysis in which the effects of material yielding are accounted for. The different types of inelastic methods may be generalized in to the following three main groups.

- Plastic zone methods.
- Elastic- plastic hinge method.
- Refined plastic hinge method.

The above generalization is based on the degree of refinement in the representation of yielding effects. The elastic plastic hinge method is the simplest approach whereas the plastic zone method is an improvement over the elastic -plastic hinge method and hence requires less computational effort and less costly (in terms of computer time) then plastic zone method.

This analytical approach combines the same principles of second-order analysis discussed previously with the plastic hinge analysis. This category of analysis is more complex than any of the other methods of analysis discussed above. It does, however, yield a more complete and accurate picture of the behavior of the structure, depending on the completeness of the model that is used. This type of analysis is often referred to as “advanced analysis.” The load-displacement curve for a second-order inelastic analysis is shown in Figure 3.1.

The second order inelastic methods are take in to account the material properties, residual stress, geometric imperfection, second order effects, three-dimensional effect, erection tolerances, and interaction with foundation.

Thus, advanced analysis methods incorporate both strength and stability behaviour in such a way that separate member design is not required.

They directly assess the strength and stability of the overall system, including interaction of the member strength and stability.

In summary, it can be seen that as more realistic and hence more complex behavior is taken into account in the analysis, the predicted critical load level is reduced or the calculated lateral displacement for a given load is increased. Thus, designers need to be aware of the assumptions utilized in any analytical approach that they employ.

3.3 Analysis of frame

The frame shown in Figure 3.2 is used to demonstrate the difference between the results of a first-order elastic analysis and a second order elastic analysis. Two kinds of analysis were carried out using STAAD.ProV8i[18] including axial, flexural and shearing deformation.
For analysis, load case 1.2 vertical(DL) + 1.6 Horizontal load(WL) is considerd. Nagetive(-ve) sign is indicate as a tension in the member.

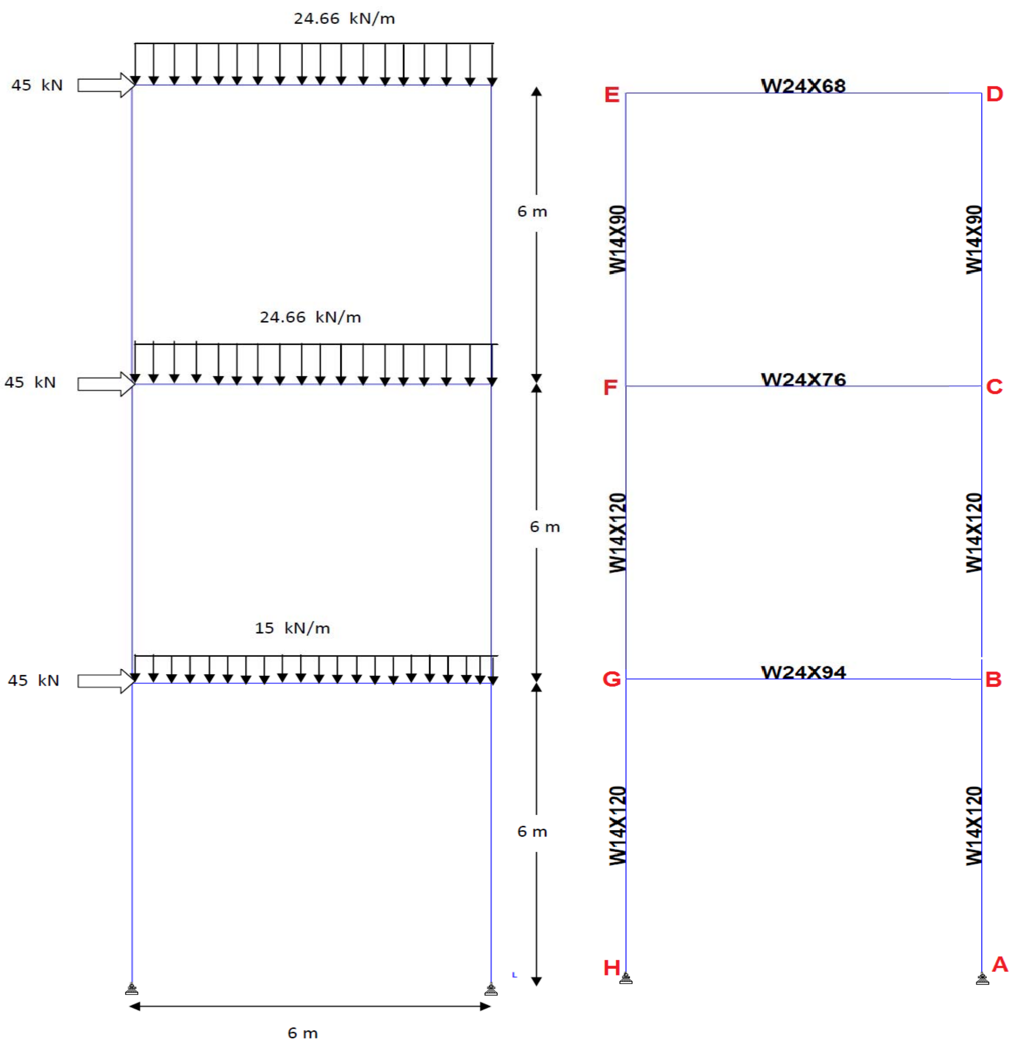


Figure 3.2: One-Bay Moment Resisting Frame

3.4 Results and discussion

The analysis results has been shown in terms of bending moment and axial force in Table 3.1. At point 'D' first order analysis deflection is 143.42 mm and second order analysis deflation is 147.96 mm. Results shows 3% increase of deflection at point D. At bottom storey (level AB) the bending moment is increased by 4% whereas at top storey (level CD) moment is increased by 1% to 2%. Axial force also changes due to

Table 3.1: Results of Analysis

Member	point	First Order elastic analysis	Second order elastic analysis	First Order elastic analysis	Second order elastic analysis
		Bending moment (kN m)	Bending moment (kN m)	Axial Force (kN)	Axial Force (kN)
Column AB	A	0	0	698.13	709.7
	B	661	687		
Column BC	B	-286	-291	363.18	365.55
	C	209	210		
Column CD	C	176	-177	143.16	144.027
	D	129	130		
Column HG	H	0	0	-153.27	-164.84
	G	-635	-661		
Column GF	G	151	153	51.18	48.82
	F	-218	-224		
Column EF	E	52.5	53	60.8	59.61
	F	74	-75.7		
Beam GB	G	786	-814	27.28	21.54
	B	869	897		
Beam FC	F	271	-277	31.51	30.77
	C	414	420		
Beam ED	E	74	-75.7	50.85	50.72
	D	176	177		

second order effect.

3.5 Summary

In this chapter various analysis types i.e. first order elastic analysis, second order elastic analysis, first order inelastic analysis and second order inelastic analysis methods has been explained. Illustration example of 3 storey 1 bay frame is presented. Due to second order effect there is increase in connection forces in the structure by around 3% also the lateral displacement. All the result has been computed using STAAD Pro V8i (20.07.07.19)[18] version.

Chapter 4

Effective Length Factor K

4.1 General

Effects of the stability on different types of frame elements such as compression members, beams, bracing system and connections and also their of the frame has been studied and as a result, several methods have been proposed for evaluating the frame strength. However, the effective length concept for evaluating the frame strength is the most popular method for estimating the interaction effects of a framed member on the total frame stability. So, K was introduced by AISC in their ASD (allowable stress design) design philosophy. K was introduced as a mathematical simplification for the column with the different joint conditions. It is dependent on several factors such as structural shape, member geometry and relative dimensions, framing members and load distribution.

In this chapter, brief history about K factor and importance of K factor in stability of structure are presented. This chapter describes about relationship between K, second order analysis and beam column strength equation. Calculation of the K factor using different method such as AISC alignment chart method, lemessurier's method, Lui's method, IS:800-1984 provision and IS:800-2007[3] provision is explained.

4.2 Introduction to K-Factor

If the axial strength of the column needs to be found out, it is necessary to calculate the buckling load of the column. Buckling load is defined as the critical load at which an ideal column became unstable due to it's slenderness. This phenomenon is called as buckling. The buckling load is the direct axial load with no transverse load to cause bending. This buckling was first studied by Euler hence it is also called as Euler's buckling or elastic buckling. This is applicable mainly to "non sway" long columns. Typically the equation for buckling load is given by,

$$P = \frac{\pi^2 EI}{KL^2} \tag{4.1}$$

K = effective length factor
EI= flexural rigidity of column
L = length of column

Current structural design practices recognize that the maximum strength of frames and the maximum strength of component members are interdependent, but it is not practical to take this interdependence into account rigorously. Structural stability research council (SSRC) technical memorandum which states that "in design practice, the two aspects, stability of individual members and elements of the structure and stability of the frame system as a whole, be considered independently"[7].

For evaluate K factor of column, multiple curve have been prepared. These curves account for the influence of residual stresses, cross-sectional shapes and imperfections on column strength. The effective length concept can be considered to relate multiple column curves to framed columns for which the amount of rotational and translational restraint provided at the ends by other members of the frame cannot be assessed accurately by simple means.

According to above concept, the strength of a framed compression member of length

L is equated to an equivalent pin ended member of length KL, subjected to axial load only.
The effective length concept is considered to be an essential part of many analysis procedures and it can handle several cases which can occur practically in all structures. The concept is valid for ideal structures, but its implementation involves several assumptions.[16]

4.3 Stability Concepts and Importance of K

The strength equations to be used are depending upon the type of analysis. If P Delta analysis is done, then the moment magnification factors are not required to be used in strength equations.
For beam column design, following points need to be used:
Find the axial strength with due considerations of practical column (Out of plumbness, residual stresses etc.). These can be taken care either at analysis stage or design stage. If these are not taken care in analysis stage then we have to use K and in-elastic strength equations to account for those.

Second order moments: Second order moments can be taken care at the time of analysis (by doing P-delta) or at the design stage by using moment magnification factor. The analysis shall include both member and sway P-delta effects.

K is just a mathematical adjustment to reduce the capacity of ideal column (for which K=1.0) to take into account the practical imperfections (To account for sway buckling).

Only doing P-delta will not serve the purpose of reducing the value of K to 1.0. If we want to really reduce K to 1.0, then we have to change the normal analysis and use some conditions which resemble the practical considerations of column, that means to develop practical situations at the analysis time to get the real effect of sway buckling and residual stresses.

To summaries,

- K is a mathematical adjustment factor which comes into picture because that application of Euler's theory to sway frames.
- The sway buckling is due to the fact that geometrical imperfections exist in the practical columns. The geometrical imperfections will be due to out of straightness, out of plumbness and other fabrication-erection tolerances.
- K is a factor used to reduce the capacity of the ideal column to account for geometric imperfections (To account for sway buckling).
- K is used at the design stage in strength equations.
- K is used to determine the axial capacity of the column (not bending capacity).
- K is always discussed with regard to buckling and not bending.

In broad perspective, the P-delta and K concepts are not directly linked. P-delta is used to take care of the bending portion of the member while the K is used to take care of the axial portion.

4.4 Methods to Calculate the Effective Length Factor K

Factor-K is the ratio between the effective length and the unbraced length of the member.

The development and implementation of effective length factors have undergone several stages. A number of methods have been proposed and these proposed methods predict K values which, when used in frame design, produce results of varying degrees of accuracy depending upon the geometry, size, support conditions and applied loading. This is due to the assumptions and simplifications made in different methods.

(a) Alignment Chart Method(AISC 360-05) The model used for the determination of K for a column braced against side sway is shown in Figure 4.1. The column under consideration is denoted as c2 in Figure 4.1. The following assumption

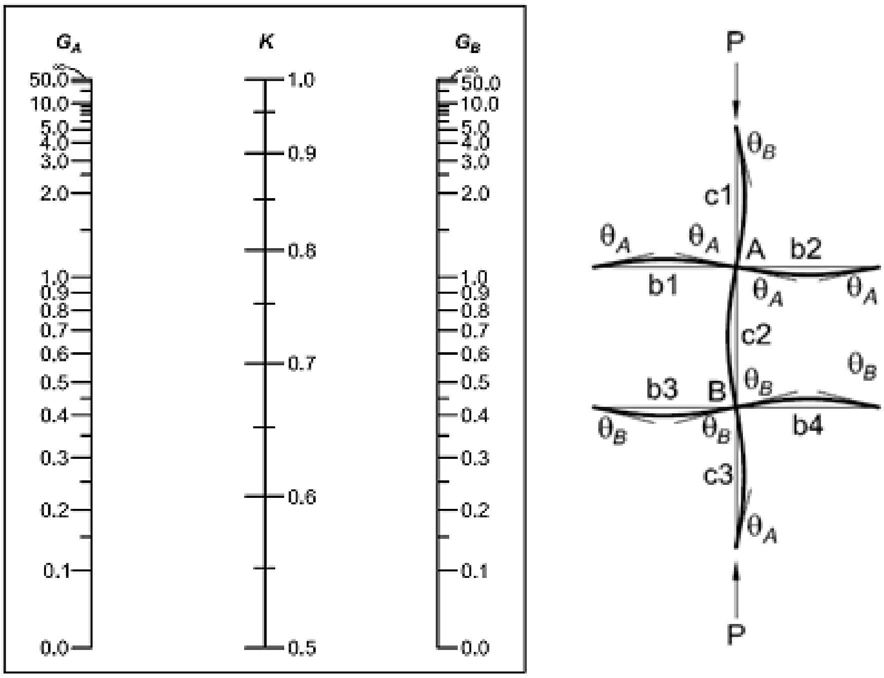


Figure 4.1: Alignment chart-sidesway inhibited (braced frame)
[1]

are made in derivation of K factor.

a. All members are prismatic and behave elasticity.

b. The axial forces in the beam are negligible.

c. All columns in a storey buckle simultaneously.

d. At a joint, the restarting moment provided by the beams is distributed among all the columns in the proportion to their stiffness.

e. At buckling, the rotation at the near and far ends of the beam equal and opposite.

f. The frame is subjected to vertical loads, applied only at the joints.

$$\frac{G_A G_B}{4}\left(\frac{\pi}{K}\right)^2 + \left(\frac{G_A G_B}{2}\right)\left(1 - \frac{\pi/K}{\tan(\pi/K)}\right) + \frac{2\tan(\pi/K)}{\pi/K} - 1 = 0 \tag{4.2}$$

Where,

$$G_A = \frac{\sum(I/L)_c}{\sum(I/L)_b} = \frac{\sum column \quad stiffness \quad metting \quad at \quad joint \quad A}{\sum beam \quad stiffness \quad metting \quad at \quad joint \quad A} \tag{4.3}$$

$$G_B = \frac{\sum(I/L)_c}{\sum(I/L)_b} = \frac{\sum column \quad stiffness \quad metting \quad at \quad joint \quad B}{\sum beam \quad stiffness \quad metting \quad at \quad joint \quad B} \tag{4.4}$$

The solution of to equation is expressed in a nomograph from(alignment chart) in Figure 4.1. Values of G_A and G_B are given on two outside scales and for K on the middle scale. The line joining G_A and G_B intersects the middle scale and will fetch the required value of K.

(b) Fames in which side sway is not prevented The modal for a column in a frame subjected to side sway is shown in Figure 4.2. The column under consideration is denoted as c2 in Figure 4.2 the assumption used in this model are the same as those used for the model of braced frame, except assumption-(e) [7]. Which is assumed as, buckling the rotations at the near and far ends of the beams are equal in magnitude and direction.

For unbraced frame

$$\left(\frac{G_A G_B(\pi/K)^2 - 36}{6(G_A + G_B}\right) - \frac{2(\pi/K)}{\tan(\pi/K)} = 0 \tag{4.5}$$

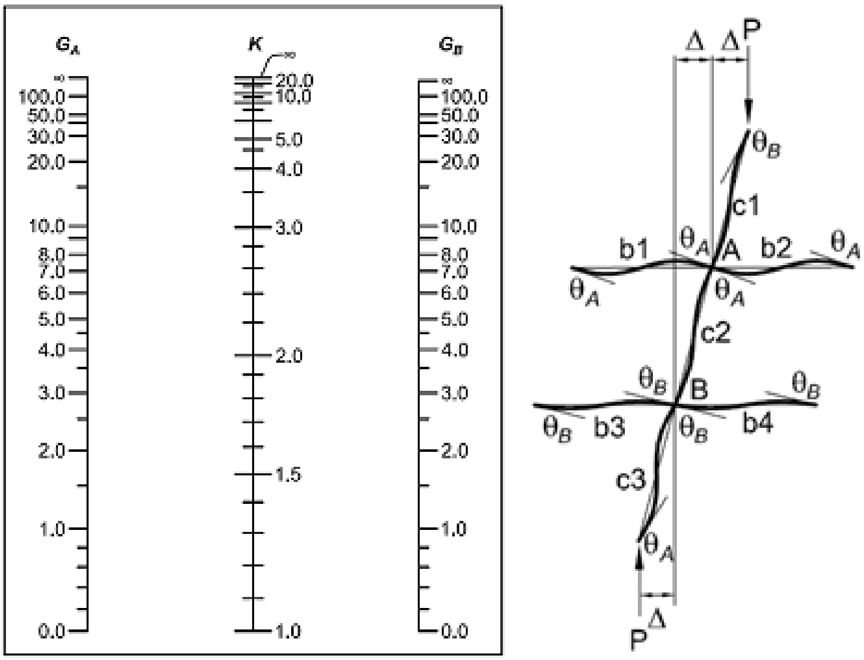


Figure 4.2: Alignment chart-sidesway uninhibited (moment frame) [1]

4.4.1 Lemessurier's Method

A more accurate method to compute K factors was given by Lemessurier[16], who proposed an approach in which the lateral restraining effect between columns can be accounted for. This approach accounts for the fact that all columns in a story buckle simultaneously, that a strong column or a column with low axial force will brace a weak column or a column carrying high axial load, or that some columns lean on others in the same story. The effective length factor for column 'i' of a story in accordance with Lemessurier, can be obtained by using the expression.

$$K_i^2 = \frac{\pi^2 EI}{P_{ui}} \left[\frac{\Sigma P_u + \Sigma C_l P_U}{\Sigma P_L} \right] \tag{4.6}$$

EI_i= flexural rigidity of column 'i'.

L_i = actual height of column 'i'.

P_{ui} = required axial compressive strength for i^{th} rigid column.

P_u= required axial compressive strength of all columns in a story.

K = K factor obtained from the sidesway permitted alignment chart.

$$P_L = \frac{\beta EI}{L^2} \tag{4.7}$$

$$\beta = \frac{6(G_A + G_B) + 36}{2(G_A + G_B) + G_A G_B + 3} \tag{4.8}$$

$$C_L P = \left(\beta \frac{K^2}{\pi^2} - 1\right) P \tag{4.9}$$

Equation accounts directly for leaner columns sized for gravity loads only. A conservative and simple design approximation using a modified elastic effective length factor K given by and suggested in the revised AISC LRFD Manual.

$$K_i^2 = \frac{I_i}{P_{ui}} \frac{\sum P_u}{I / \sum K^2}, P_{ui} > 0 \tag{4.10}$$

4.4.2 Lui's Method

A simple and elegant method which accounts for both member instability and frame instability in the calculation of effective length factors was proposed recently by Lui[16]. Member instability, referred to as the P-delta effect is considered in terms of stability functions which are simplified to a great extent by using a Taylor series expansion. Frame instability, referred to as the P-delta effect, is accounted for by the use of a story stiffness concept. The two effects are explicitly combined into one formula, for which K factor for a member 'i' in a frame can be determined as

$$K_i^2 = \frac{\pi^2 EI_i}{P_i L_i^2} \sum \frac{P}{L} \left(\frac{1}{5 \sum \eta} + \frac{\Delta_{oh}}{\sum H} \right) \tag{4.11}$$

P = Compressive axial force in member.

$\sum (P/L)$= sum of the axial force to length ratio of all members in a story.

$\sum H$= sum of the story lateral forces at and above the story under consideration.

Δ_{oh}= Inter-story deflection i.e. relative displacement between adjacent stories.

$\eta = \frac{(3+48m+4.2m^2)EI}{L^3}$

η=member stiffness index.

$m = M_A/M_B$

M_A and M_B are Member end moments.

$M_A < M_B$

$\sum \eta$= sum of h of all members in the story being considered.

4.4.3 IS:800-1984 Method

In the absence of more exact analysis, the effective length of columns in framed structures may be obtained from the ratio I/L, of effective length l to unsupported length L given in Figure 4.3a when relative displacement of the ends of the column is prevented and in Figure 4.3b when relative lateral displacement of the ends is not prevented. In the later case, it is recommended that the effective length ratio l/L may not be taken to be less than 1.2.

In Figure 4.3, β_1 and β_2 are equal to,

$$\frac{\sum K_c}{\sum K_c + \sum K_b}$$

Where the summation is to be done for the members framing into a joint at top and bottom respectively K_c and K_b being the flexural stiffnesses for the column and beam, respectively.

4.4.4 IS:800-2007 Method

the code(IS:800-2007), gives the following equation for the effective length factor K based on woods curve: For non sway frames(braced frame):

$$K = \frac{[1 + 0.145(\beta_1 + \beta_2) - 0.265\beta_1\beta_2]}{[2 - 0.364(\beta_1 + \beta_2) - 0.247\beta_1\beta_2]} \tag{4.12}$$

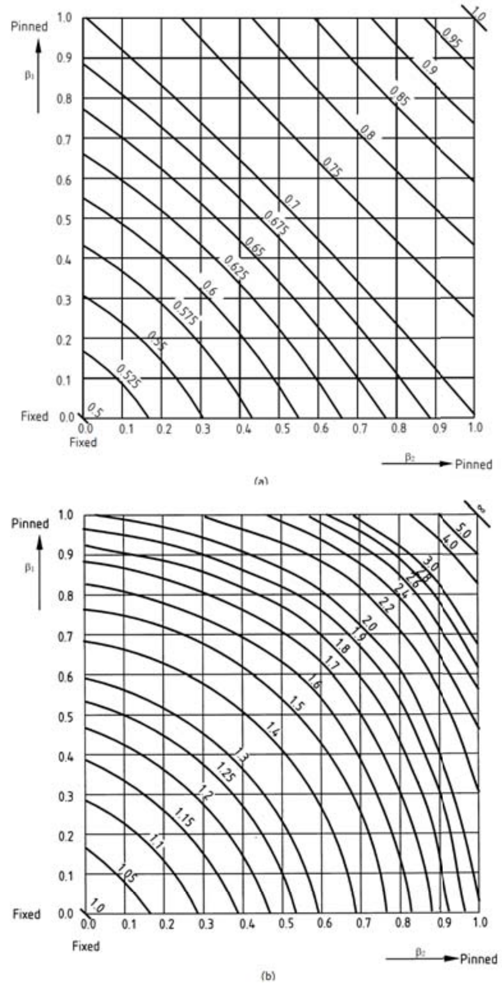


Figure 4.3: Effective length ratio for column in a frame (a) Non Sway Frame (b) Sway Frame

[5]

For sway frames(moment resisting frame):

$$K = \left\{ \frac{[1 - 0.2(\beta_1 + \beta_2) - 0.12\beta_1\beta_2]}{[1 - 0.8(\beta_1 + \beta_2) - 0..6\beta_1\beta_2]} \right\}^2 \tag{4.13}$$

$$\beta_i = \sum K_c / (\sum K_c + \sum K_b) \tag{4.14}$$

Where K_C and $K_B$are the effective flexure stiffness of the column and beams meeting at the joint at the ends of column and rigidly connected at the joint. K_C or K_B= C(I/L) Where I is the moment of inertia about an axis perpendicular to the plan of the structure frame, L is length of the member. Taken as center to center distance of the frame, L is the length of intersecting member and 'C' is the connection factor as shown in table 4.1 $\bar{\eta}= P/P_{cr}$, Where P is the applied load and P_{cr} is the effective

Table 4.1: connection factor C

Fixity condition	**Connection Factor C**	
	Braced Frame	Unbraced frame
Pinned Connection	1.5(1-$\bar{\eta}$)	0.5(1-$\bar{\eta}$)
Rigidly connected column	1.0(1-$\bar{\eta}$)	1.0(1-0.2$\bar{\eta}$)
Fixed	2.0(1-0.4$\bar{\eta}$)	0.67(1-0.4$\bar{\eta}$)

buckling load =$\pi^2 EI/(KL)^2$

Note that for calculating C it need the effective length and hence the determination of effective length is an interactive process. Initially, we can assume K =1 for calculating the value of C.

4.5 An Illustrative Example

The following example of a frame with a leaner column illustrates the computation of K factors by using the four methods described above. The frame, which was considered as shown in Figure 4.4

The K factor for the right column AB is evaluated as follows:

Data assumed for example: both column and beam have same section ISMB 400, gravity load at point B and C applied as P= 50 kN, and a small lateral load of the gravity loads(1P% of 100 kN) viz. 1 kN at point C as shown in Figure 4.4.

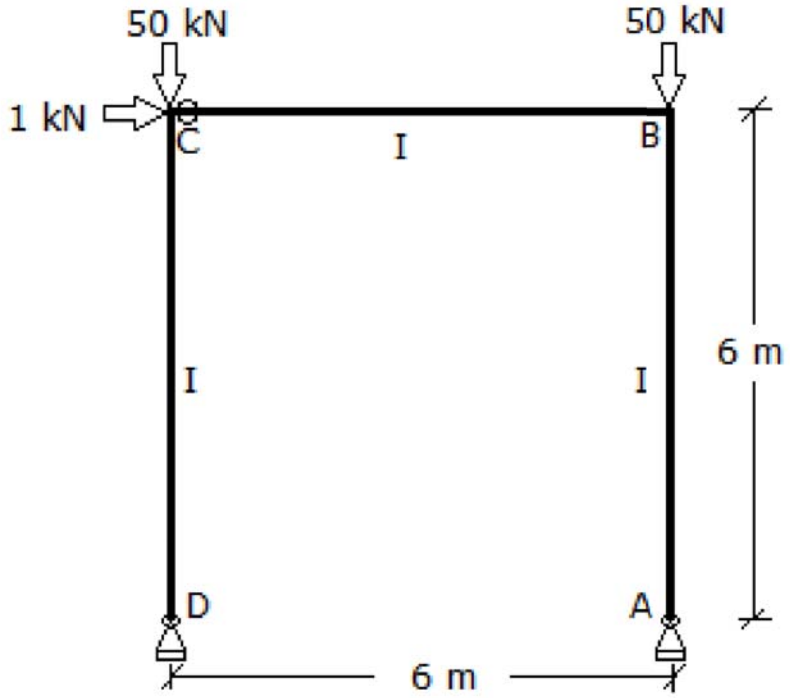


Figure 4.4: Illustrative Example of a Leaned Column

Table 4.2: Input Data

Column	I mm^4	L mm	P (N)	P/L	M	η
AB	20458.4×10^4	6000	50×10^3	8.33	0	$\frac{3EI}{L^3}$
CD	20458.4×10^4	6000	50×10^3	8.33	0	$\frac{3EI}{L^3}$
$\sum$			100×10^3	16.66	0	$\frac{6EI}{L^3}$

4.5.1 Alignment Chart Method

$$G_A = \frac{\sum\left(\frac{I}{L}\right)_{column}}{\sum\left(\frac{I}{L}\right)_{beam}} = \infty$$

$$G_A = \infty$$

$$G_B = 2\frac{\sum\left(\frac{I}{L}\right)_{column}}{\sum\left(\frac{I}{L}\right)_{beam}} = 2.0$$

$$\mathbf{K = 2.6} \tag{4.15}$$

4.5.2 Lemessuriers Method

For this frame, since only Column AB provides stability to the system,

$$K_{AB}^2 = \frac{\pi^2 EI}{P_{AB}L^2}\left[\frac{\Sigma P_u + \Sigma C_L P_U}{\Sigma P_L}\right] \tag{4.16}$$

EI_i= flexural rigidity of column 'i'.

L_i = actual height of column 'i'.

P_{ui} = required axial compressive strength for i_{th} rigid column.

P_u=required axial compressive strength of all columns in a story.

K = K factor obtained from the sidesway permitted alignment chart.

$$P_L = \frac{\beta EI}{L^2}$$

$$\beta = \frac{6(G_A + G_B) + 36}{2(G_A + G_B) + G_A G_B + 3}$$

$$C_L P = \left(\beta \frac{K^2}{\pi^2} - 1\right) P$$

K = K factor obtained from the sidesway permitted alignment chart.

so,

$$\beta = \frac{6(G_A + G_B) + 36}{2(G_A + G_B) + G_A G_B + 3}$$

$$\beta = \frac{6+6}{2+2} = 1.5$$

$$(C_L P)_{AB} = \left(\beta_{ab} \frac{K^2}{\pi^2} - 1\right) P$$

$$(C_LP)_{AB} = \left(1.5\frac{2.6^2}{\pi^2} - 1\right)P$$
$$(C_LP)_{AB} = 0.027P$$

$$K_{AB}^2 = \frac{\pi^2 EI}{P_{AB}L^2}\left[\frac{\Sigma P_u + \Sigma C_L P_U}{\Sigma P_L}\right]$$

$$P_L = \frac{\beta EI}{L^2}$$

$$K_{AB}^2 = \frac{\pi^2 EI}{P_{AB}L^2}\left[\frac{2P + 0.027P}{\beta\frac{EI}{L^2}}\right]$$

$$K_{AB}^2 = 13.44$$

$$\mathbf{K = 3.65} \tag{4.17}$$

4.5.3 Lui's Method

$$K_{AB} = \frac{\pi^2 EI}{PL^2}\sum\frac{P}{L}\left(\frac{1}{5\sum\eta} + \frac{\Delta_{oh}}{\sum H}\right)$$

P = compressive axial force in member AB.

$\sum(P/L)$= sum of the axial force to length ratio of all members in a story.

$\sum H$= sum of the story lateral forces at and above the story under consideration.

Δ_{oh}= Inter-story deflection i.e. relative displacement between adjacent stories.

$\eta = \frac{(3+48m+4.2m^2)EI}{L^3}$

η =member stiffness index.

$m = M_A/M_B$

$M_A, M_B = memberendmomentswithM_A < M_B$

η = sum of h of all members in the story being considered.

$$K^2 = \frac{\pi^2 \times 2\times10^5 \times 20458.4\times10^4}{50\times10^3\times6000^2}\left[16.66\left[\frac{1}{5\times6\times\frac{2\times10^5\times20458.4\times10^4}{6000^3}} + \frac{3.488}{1000}\right]\right]$$

$$K^2 = 13.76$$

$$\mathbf{K = 3.71} \tag{4.18}$$

4.5.4 IS:800-1984 specification

$I_b = I_c$ =20458.4×10^4 mm^4

$K_c = I_c/L_c$ K_c =20458.4 × 10^4 /6000 = 34.09 × 10^3 mm^3

$K_b = I_b/L_b$ K_b =20458.4 × 10^4 /6000 = 34.09 × 10^3 mm^3

$$\mathbf{K = 2.49} \tag{4.19}$$

4.5.5 IS:800-2007 specification

$$\Sigma K_C = C(I_c/h_s)$$

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

$$P_{cr} = \frac{\pi^2 \times 2 \times 10^5}{6000^2} = 11.22 \times 10^6 N$$

$$\bar{n} = \frac{P}{P_{cr}} = \frac{50 \times 10^3}{11.22 \times 10^6} = 0.00445$$

Fixity condition at far end is pinned:

$$C = 0.67(1 - \bar{n})$$
$$C = 0.67(1 - 0.00445)$$
$$C = 0.668$$

$$\Sigma K_c = \Sigma K_b = 0.668 \times \frac{20458.4 \times 10^4}{6000} = 22770mm^3$$

$$\Sigma K_b = \frac{20458.4 \times 10^4}{6000} = 34097mm^3$$

$$\beta_2 = \frac{22770}{22770 + 34097} = 0.4$$

$$\beta_1 = \frac{22770}{22770 + 0} = 1$$

$$\beta_1 = 1$$

$$K = \left[\frac{(1 - 0.2(\beta_1 + \beta_2) - 0.12\beta_1\beta_2)}{(1 - 0.8(\beta_1 + \beta_2) + 0.6\beta_1\beta_2)}\right]^{0.5}$$

$$\mathbf{K = 2.37} \tag{4.20}$$

4.6 Result and Discussion

Table 4.3: Results of K factor

Method	**AISC (Alignment chart)**	**Lemessurier**	**Lui's**	**IS:800-1984**	**IS:800-2007**
K factor	2.6	3.65	3.71	2.49	2.37

Result obtained from different method has been presented in Table 4.3. It shows that the Lui's methods and Lemessurier's methods gives conservative result where as one basic difference between IS:800-1984[5] and IS:800-2007 [3] is application of connection factor "C" depending upon joint condition at far end in the later case.

4.7 Summary

Five different approaches, including the alignment chart, LeMessurier's formula, Lui's formula, IS:800-1984 and the IS:800-2007 are considered to compute K factors of columns in frames. Out of this five the lui's methods and Lemessurier Method gives most conservative result.

For calculating effective length one basic difference in earlier IS:800-1984[5] and new IS:800-2007[3] is connection factor "C" which is dependent on connection condition such as pinned connection or rigidly connected column or fixed connection.

Chapter 5

AISC:360-2005 : Specification for Stability Design

5.1 General

This chapter explains steel structure stability requirements laid out by American national standard ANSI/AISC 360-05 “specification for steel building” which is a commonly referred design specification by the engineering industry.

The-13th edition of AISC[1] (American institute of steel construction) 2005 specification for steel structure provide an integrated treatment of allowable stress design (ASD) and load and resistance factor design(LRFD) specification for new stability analysis and design criteria for steel structure in Chapter C[1].

The stability of structures must be considered from the standpoint of the structure as a whole, including not only the compression members, but also the beams, bracing systems and connections. Various methods are available to provide stability[1].

Chapter C of AISC:360-05 specifies that the design of the structure for stability must consider all of the following:

a. Flexural, shear, and axial deformations of members.

b. All component and connection deformations that contribute to the lateral displacement of the structure.

c. P-Δ effects, which are the effects of loads acting on the displaced location of joints or nodes in the structure.

d. P-δ effects, which are the effects of loads acting on the deformed shape of a member between joints or nodes.

e. Geometric imperfections, such as initial out-of-plumbness.

f. The reduction in member stiffness due to residual stresses and, in particular, the effect of this stiffness reduction on the stability of the structure.

The 2005 AISC[1] Specification offers three alternatives for the design of structures for stability:

a. First-Order Analysis Method in Section C2.2b.

b. The Second-Order analysis method in Section C2.2a.

c. More rigorous analysis method prescribes in Appendix 7 (AISC 360-05) the direct analysis method.

5.2 Specification for Stability Design

Chapter C of AISC 360-05 describe the following requirement.

a. Stability Design Requirements.

b. Calculation of Required Strengths.

5.3 Stability Design Requirements

Design requirement shall be provided for the structure as a whole and for each of its elements. All the effects mention in section 5.1 are to be considered on the stability of the structure and its elements for design. The Specification addresses traditional approach, termed as the Effective Length Method, and new approach which is termed as the direct analysis method, addressed in Appendix 7(AISC 360-05[1]).

In either the Effective Length or the direct analysis method, structural analysis by itself is not sufficient to provide for the stability of the structure as a whole. The overall stability of the structure as well as the stability of individual elements is provided for by the combined calculation of the required strengths by structural analysis and the satisfaction of the member and connection design provisions of the Specification.

In general, it is essential that an accurate second-order analysis of the structure be performed.

5.4 Calculation of Required Strengths

5.4.1 Methods of Second-Order Analysis

Second-order analysis shall conform to the following requirements.

(a) General Second-Order Elastic Analysis

Any second-order elastic analysis method that considers both P-Δ and P-δ effects may be used otherwise the amplified first-order elastic analysis method defined in Section 5.4.1.b is an accepted method for second-order elastic analysis of braced, moment, and combined framing systems.

(b) Second-Order Analysis by Amplified First-Order Elastic Analysis

The following is an approximate second-order analysis procedure for calculating the required flexural and axial strengths in members of lateral load resisting systems. The required second-order flexural strength, M_r, and axial strength, P_r, shall be determined as follows:

$$M_r = B_1 M_{nt} + B_2 M_{lt} \tag{5.1}$$

$$P_r = P_{nt} + B_2 P_{lt} \tag{5.2}$$

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1 \tag{5.3}$$

$$B_2 = \frac{1}{1 - \frac{\alpha \sum P_{nt}}{\sum P_{e2}}} \geq 1 \tag{5.4}$$

M_r = required second-order flexural strength.
M_{nt} = first-order moment, assuming there is no lateral translation of the frame.
M_{lt} = first-order moment caused by lateral translation of the frame only.
P_r = required second-order axial strength.
P_{nt} = first-order axial force, assuming there is no lateral translation of the frame.
$\sum P_{nt}$ = total vertical load supported by the story, including gravity column loads.
P_{lt} = first-order axial force caused by lateral translation of the frame only.
C_m = a coefficient assuming no lateral translation of the frame whose value shall be taken as follows:

a. For beam-columns not subject to transverse loading between supports in the plane of bending,

$$C_m = 0.6 - 0.4(M_1/M_2) \tag{5.5}$$

Where, M_1 and M_2, calculated from a first-order analysis, are the smaller and larger moments, respectively.

b. For beam-columns subjected to transverse loading between supports, the value of C_m shall be determined either by analysis or conservatively taken as 1.0 for all cases.

P_{e1} = elastic critical buckling resistance of the member in the plane of bending, calculated based on the assumption of zero sidesway.

$$P_{e1} = \frac{\pi^2 EI}{K_1 L^2} \tag{5.6}$$

$\sum P_{e2}$ = elastic critical buckling resistance for the story determined by sidesway buckling analysis.

For moment frames, where sidesway buckling effective length factors K_2 are determined for the columns, it is permitted to calculate the elastic story sidesway buckling resistance as

$$\Sigma P_{e2} = \Sigma \frac{\pi^2 EI}{(K_2 L)^2} \tag{5.7}$$

For all types of lateral load resisting systems, it is permitted to use

$$\Sigma P_{e2} = R_M \frac{\Sigma HL}{\Delta_H} \tag{5.8}$$

Where,

E = modulus of elasticity of steel.

R_M = 1.0 for braced-frame systems;

R_M = 0.85 for moment-frame and combined systems.

I = moment of inertia in the plane of bending.

L = story height.

K_1 = effective length factor calculated based on the assumption of no lateral translation.

K_2 = effective length factor in the plane of bending, calculated based on a sidesway buckling analysis.

Δ_H = first-order interstory drift due to lateral forces.

$\sum_H$ = story shear produced by the lateral forces used to compute Δ_H.

5.4.2 Design Requirements

These requirements apply to all types of braced, moment, and combined framing systems.

(a) Design by second- order analysis

Design by second-order analysis is essentially the traditional effective length method with an additional requirement for a minimum lateral load. It is permitted when the ratio of second-order drift, Δ_{2nd}, to first-order drift, Δ_{1st}, is equal to or less than 1.5, and requires the use of:

a. A explicit direct second-order analysis or a first-order analysis with $B_1 - B_2$ amplification.

b. The nominal frame geometry with a minimum lateral load ("notional load") $N_i = 0.002Y_i$, where Y_i is the total gravity load on level i from LRFD load combinations.

c. The nominal stiffnesses EA and EI.

When the ratio of second-order drift to first-order drift, which is given by B_2, is equal to or less than 1.1, $K = 1.0$ can be used in the design of moment frames. Otherwise, for moment frames, K is determined from a sidesway buckling analysis.

(b) Design by First-Order Analysis

This section provides a method for designing frames using a first-order elastic analysis with K = 1.0, provided the sidesway amplification $\Delta_{2nd}/ \Delta_{1st} \leq 1.5$.
The first-order analysis method is permitted when:

a. The ratio of second-order drift,Δ_{2nd}, to first-order drift,Δ_{1st}, is equal to or less than 1.5.

b. The column axial force $\alpha P_r \leq 0.5P_y$, where $\alpha = 1.0$ for LRFD, 1.6 for ASD.

This method requires the use of:

- A first-order analysis.
- The nominal frame geometry with an additional lateral load $N_i = 2.1(\Delta/L)Y_i \leq 0.0042Y_i$, applied in all load cases.
- The nominal stiffnesses EA and EI.
- B_1 as a multiplier on the total moment in beam columns.

For all frames designed with this method, $K = 1.0$.

(c) Design by Direct Analysis (Appendix 7)

The direct analysis method, addresses a new method for the stability analysis and design of structural steel systems comprised of moment frames, braced frames, shear walls or combinations thereof. While the precise formulation of the method is unique to the AISC Specification, some of its features have similarities to other major design specifications around the world including the Eurocodes, the Australian Standard, the Canadian Standard and ACI 318.

The direct analysis method addresses the influence of nominal geometric imperfections (for example, out-of-plumbness) and stiffness reductions due to distributed yielding directly within the analysis.

This specification can be applied to structural systems comprised of moment frames, braced frames, shear walls, or combinations thereof.
following are the requirement for direct analysis.

1. General Requirements
2. Notional Loads
3. Design-Analysis Constraints

1. General requirements

General requirement Members shall satisfy the provisions of Section H1 (Members Subject to Flexure and axial force) of AISC 360-05[1] with the nominal column strengths, P_n, determined using $K = 1.0$.

2. Notional loads

These are applied on the structure to account for the effects of geometric imperfections, inelasticity, or both. Notional loads are lateral loads that are applied at each framing level and specified in terms of the gravity loads. Notional loads shall be applied in the direction that adds to the destabilizing effects under the specified load combination.

The purpose of notional loads is to account for the destabilizing effects of geometric imperfections, non-ideal conditions (such as incidental patterned gravity load effects, temperature gradients across the structure, foundation settlement, uneven column shortening, or any other effects that could induce sway that is not explicitly considered in the analysis), inelasticity in structural members, or combinations thereof. To accounts any or all of these potential effects, the magnitude of the notional load $0.002\mathrm{Y}_i$ can be thought of as representing an initial out-of-plumbness in each story of the structure of 1/500 times the story height.

3. Design-analysis constraints

The amplification of first-order analysis is an approximate second order elastic analysis. Where stability effects are significant, consideration must be given to initial geometric imperfections in the structure due to fabrication and erection tolerances.

1. The second-order analysis shall consider both P-Δ and P-δ effects. It is permitted to perform the analysis using any general second-order analysis method, or by the amplified first-order analysis method, provided that the B_1 and B_2 factors are based on the reduced stiffnesses.

2. A notional load, $N_i = 0.002Y_i$, applied independently in two orthogonal directions, shall be applied as a lateral load in all load combinations. This load shall be in addition to other lateral loads, if any, where

N_i = notional lateral load applied at level i.

Y_i = gravity load from the LRFD load combination or 1.6 times the ASD load combination applied at level i.

The notional load coefficient of 0.002 is based on assuming initial geometric imperfections. Initial imperfection conservatively equal to the maximum fabrication and erection tolerances permitted by the AISC, 2005[1]. For columns and frames, this implies a member out-of-straightness equal to L/1000, where L is the member length between brace or framing points, and a frame out-of-plumbness equal to H/500, where H is the story height.

3. A reduced flexural stiffness, EI^*,There are two reasons for imposing the reduced stiffness for analysis.

- For frames with slender members, where the limit state is governed by elastic stability, the Specification for 0.8 factor on stiffness results in a system available strength equal to 0.8 times the elastic stability limit. This is roughly equivalent to the margin of safety implied by design of slender columns by the effective length procedure where the design strength,

$\phi P_n = 0.9(0.877)P_e = 0.79P_e$
ϕP_e = elastic critical load,
0.90 = resistance factor
0.877 is a reduction factor in the column curve equation ($F_{cr} = 0.877F_e$, F_{cr} = Flexural buckling stress, F_e = Elastic critical buckling stress)

- For frames with intermediate columns, the $0.8\tau_b$ factor reduces the stiffness to account for inelastic softening. The τ_b factor is similar to the inelastic stiffness reduction factor implied in the column curve to account for loss of stiffness under high compression loads ($P_u > 0.5P_y$),and the 0.8 factor accounts for additional softening under combined axial compression and bending.

The reduction coefficients for both slender columns are close enough, such that the single reduction factor of $0.8\tau_b$ works over the full range of slenderness.

The reduced stiffness and notional load requirements apply only to the analyses for strength limit states. They do not apply to analyses of serviceability conditions of excessive deflections, vibration, etc. For ease of application in design practice, where $\tau_b = 1$, the reduction on EI and EA can be applied by modifying E in the analysis.

$$EI^* = 0.8\tau_b EI \tag{5.9}$$

where I = moment of inertia about the axis of bending.
$\tau_b = 1.0$ for $\alpha P_r/P_y \leq 0.5$
$\tau_b = 4[\alpha P_r/P_y$ (1-$\alpha P_r/P_y$)] for $\alpha P_r/P_y > 0.5$
P_r = Required axial compressive strength under LRFD or ASD load combinations.
$P_y = AF_y$, Member yield strength.
α = 1.0 (LRFD) = 1.6 (ASD)

4. A reduced axial stiffness, EA^*

$$EA^* = 0.8EA \tag{5.10}$$

It is used for members whose axial stiffness is considered to contribute to the lateral stability of the structure, where A is the cross-sectional member area.
It requires the use of

a. A direct second-order analysis or a first-order analysis with $B_1 - B_2$ amplification.

b. The nominal frame geometry with an additional lateral load of $N_i = 0.002Y_i$, where Y_i is the total gravity load on level i.

c. The reduced stiffnesses EA^* and EI^* (including in $B_1 - B_2$ amplification, if used).

d. LRFD load combinations, or ASD load combinations multiplied by 1.6. This multiplier ensures that the drift level is consistent for LRFD and ASD when determining second-order effects. The forces and moments obtained in this analysis are then divided by 1.6 for ASD member design.

The following exceptions apply as alternatives in item b:

- If the out-of-plumb geometry of the structures is used, the notional loads can be omitted.

- When the ratio of second-order drift to first-order drift is equal to or less than 1.5, the notional load can be applied as a minimum lateral load, not an additional lateral load. Note that the unreduced stiffnesses, EA and EI, are used in this comparison.

- When the actual out-of-plumbness is known, it is permitted to adjust the notional loads proportionally. For all frames designed with this method, $K = 1.0$.

4. The Simplified Method

This method is provided in the AISC Basic Design Values Cards and the 13^{th} Edition Steel Construction Manual[1], and excerpted as shown in Figure 5.1. This simplified method is derived from the effective length method using $B_1 - B_2$ amplification with B_1 taken equal to B_2.

Note that the user note in Section C2.1b[1] says that B1 may be taken equal to B_2 as long as B_1 is less than 1.5. However, it is also conservative to take B_1 equal to B_2 any time B_1 is less than B_2.

To make simplifying the assumptions this method is conservative with assumption that B_1 equal to B_2 any time and based on that as shown in Figure 5.1 the basic design value card has been obtain.

Simplified Method

Step 1. Perform first-order analysis. Use 0.2% of total story gravity load as minimum lateral load in all load combinations.
Step 2. Establish the design story drift limit and determine the lateral load required to produce it.
Step 3. Determine the ratio of the total story gravity load to the lateral load determined in Step 2. For ASD, multiply by 1.6.
Step 4. Multiply first-order results by the tabular value. *K*=1, except for moment frames when the tabular value is greater than 1.1.

Design Story Drift Limit	Ratio from Step 3 (times 1.6 for ASD, 1.0 for LRFD)										
	0	5	10	20	30	40	50	60	80	100	120
H/100	1	1.1	1.1	1.3	1.4				When ratio exceeds 1.5, simplified method requires a stiffer structure.		
H/200	1	1	1.1	1.1	1.2	1.3	1.3	1.4			
H/300	1	1	1	1.1	1.1	1.2	1.2	1.3	1.4	1.5	
H/400	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.3	1.4
H/500	1	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.3

Figure 5.1: Simplified Methods from AISC Basic Design Values Cards.

Note for use of simplified method:

a. When the ratio of second-order drift, Δ_{2nd}, to first-order drift,Δ_{1st}, is equal to or less than 1.5 as with the Design by Second-Order Analysis method. It allows the use of a first-order analysis based on nominal stiffnesses, EA and EI, with a minimum lateral load $N_i = 0.002Y_i$.

b. The ratio of total story gravity load to the story lateral load is used to enter the table in Figure 5.1. The second-order amplification multiplier is determined from the value in the table corresponding to the calculated load ratio and design story drift limit. While linear interpolation between tabular values is permitted, it is important to note that the tabular values have, in essence, only two significant digits. Accordingly, the value determined should not be calculated to more than one decimal place. The tabular value is used to amplify all forces and moments in the analysis.

c. When the ratio of second-order drift to first-order drift is equal to or less than 1.1, $K = 1.0$ can be used in the design of moment frames. Otherwise, for moment frames, K is determined from a sidesway buckling analysis. For braced frames, $K = 1.0$.

5.5 An Example: Three storey one bay frame

A moment frame has been considered for stability analysis. Loading is considered as shown in Figure 5.2. Design criteria and loading on structure is based on PIP STC01015 Structural design criteria[19] and also referred paper on design of structural steel piperacks published in engineering Journal 4^{th} quarter 2010 by AISC[20]. All the three analysis is carried out using STAAD Pro v8i[18] version (20.07.07.19) and the results are compared with manual calculation.

The geometry of frame loading and section size is consider as shown on Figure 5.2. For design load case 1.2 dead load(vertical load) + 1.6 wind load(horizontal load) is consider.

A trial shape for column AB is selected $W14 \times 120$ and corresponding drift of frame is:

$\Delta_B = 92.78$ mm

K factor for column AB is calculated as per AISC Alignment chart method. $K = 2.3$ (AISC Alignment Chart Method 4.2)

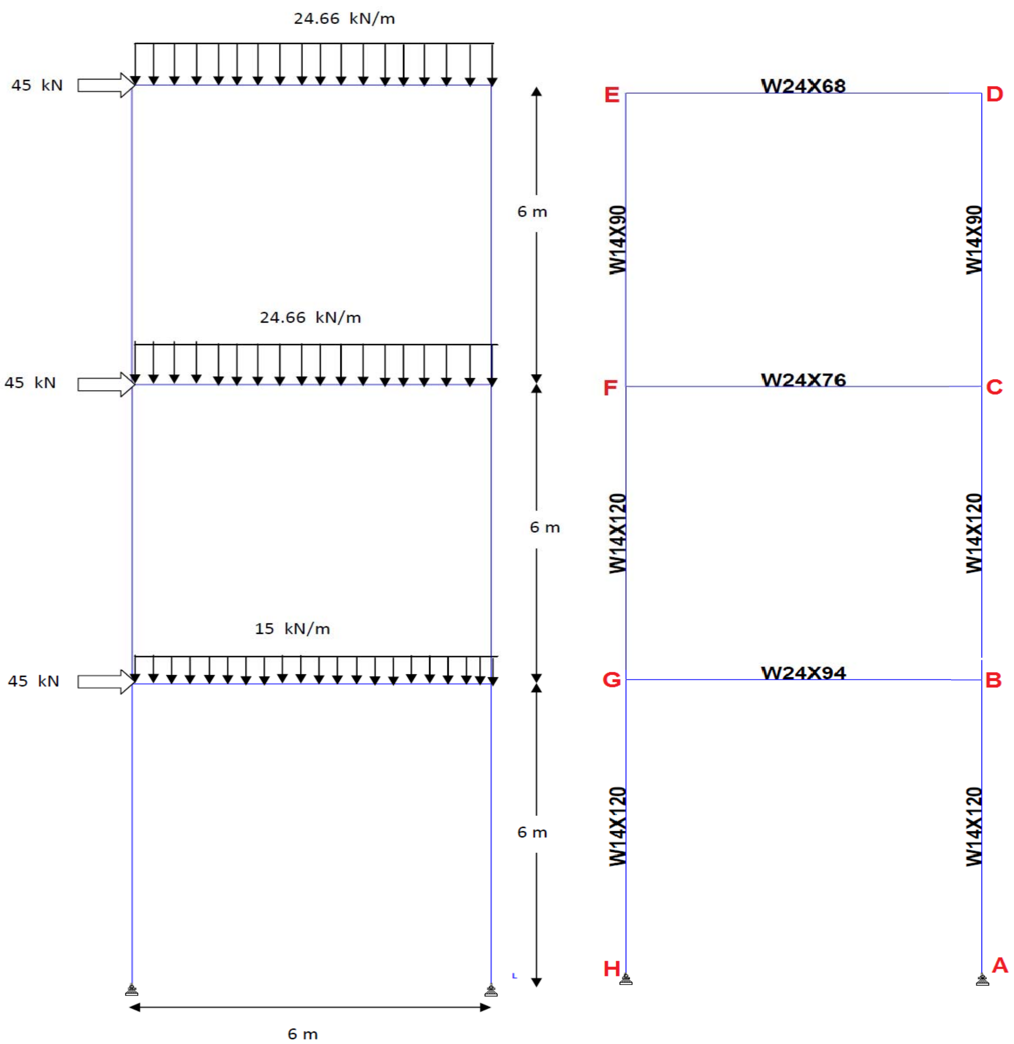


Figure 5.2: One-Bay Moment Resisting Frame

5.5.1 Design by Second-Order Analysis (section C2.2a[1])

For the example frame given in Figure 5.2, the minimum lateral load based upon the total gravity load, Y_i is:

$Y_i = 1.2(147.96 + 147.96 + 90)$

$Y_i = 463.10$ kN

$N_i = 0.002$ Y$_i$

$N_i = 0.002$ (463.10)kN

$N_i = 0.92$ kN

Because this notional load is less than the actual lateral load, it need not be applied. For a load combination that did not include a lateral load, the notional load need to be included in the analysis.

For Column AB, using first-order analysis and B_1 - B_2 amplification

$P_{nt} = 279$ kN, $P_{lt} = 432$ kN

$M_{nt} = 0$ kNm, $M_{lt} = 648$ kNm

For $P - \delta$ amplification.

$$B_1 = \frac{C_m}{1 - \alpha P_r/P_{e1}} \geq 1$$

$$C_m = 0.6 - 0.4(M_1/M_2)$$

$$C_m = 0.6$$

$$P_{e1} = \pi^2 EI/\left(KL\right)^2$$

$$P_{e1} = \pi^2 \times 2 \times 10^5 \times 1380 \times 25.4^4/\left(2.3 \times 6000\right)^2$$

$$P_{e1} = 5.95 \times 10^6\ kN$$

$$B_1 = \frac{0.6}{1 - \left(1 \times \frac{711 \times 10^3}{5.95 \times 10^6}\right)} \geq 1$$

$$B_1 = 0.68 \ngeq 1$$

$$B_1 = 1$$

For $P - \Delta$ amplification

The first-order drift ratio is determined from the calculated drift of 92.78 mm. Thus,

$$\Delta_{1st}/L = 92.78/6000 = 0.0154$$

$$\Sigma P_{e2} = R_M \sum H/\left(\Delta_{1st}/L\right)$$

$$\Sigma P_{e2} = 0.85(1.6 \times 10^3)/(0.0154)$$

$$\Sigma P_{e2} = 3.96 \times 10^6\ N$$

$$\Sigma P_{e2} = R_M \sum H / \left(\Delta_{1st}/L\right)$$

$$B_2 = \frac{1}{1 - \frac{\alpha \sum P_{nt}}{\sum P_{e2}}} \geq 1$$

$$B_2 = \frac{1}{1 - \left(\frac{1 \times 108 \times 10^3}{3.96 \times 10^3}\right)} \geq 1$$

$$B_2 = 1.028 \geq 1$$

$$B_2 = 1.028$$

Because $B_2 = 1.028$, the second-order drift is less than 1.5 times the first-order drift. Thus, the use of this method is permitted.

The amplified axial force (Equation C2-1b[1]) and associated design parameters for this method are:

$$P_r = P_{nt} + B_2 P_{lt}$$

$$P_r = 279 + 1.205(432)$$

$$P_r = 723.096\ kN$$

The amplified moment (Equation C2-1a [1]) and associated design parameters for this method are:

$$M_{rx} = B_1 M_{nt} + B_2 M_{lt}$$

$$M_{rx} = 1.0(0) + 1.028(648)$$

$$M_{rx} = 666.14\ kNm$$

Based upon these design parameters, the axial and strong axis available flexural strengths of the ASTM[21] A992 $W14 \times 120$ are:

$$\frac{KL}{r_x/ry} = \frac{2.3 \times 6}{1.67} = 8.26\ m \tag{5.11}$$

From equation 5.11 Calculate axial strength(P_c), refer AISC 360-05[1] Chapter E (Design of compression member) or form table given in AISC 360 manual[1] refer page 4-13.

$$P_n = F_{cr} A_g$$

$$P_c = \phi_c P_n$$

form AISC manual Table 4-1 page 4-13[1]

$$P_c = 4495.91 \times 10^3\ N$$

$$M_{cx} = \phi_b M_{nx}$$

$$M_{cx} = 795/0.7375 \times 10^6\ Nmm$$

$$M_{cx} = 1077.96 \times 10^6\ Nmm$$

To determine which interaction equation is applicable, the ratio of the required axial compressive strength to available axial compressive strength must be determined.

$$\frac{P_r}{P_c} = \frac{711 \times 10^3}{44499.15 \times 10^3}$$

$$\frac{P_r}{P_c} = 0.156 < 0.2$$

Thus, because $P_r/P_c < 0.2$, Equation H1-1b [1] is applicable.

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0$$

$$= 0.079 + \left(\frac{666.14 \times 10^6}{1077.96 \times 10^6}\right)$$

Interaction ratio = 0.697

5.5.2 Design by First-Order Analysis (Section C2.2b[1])

For all frames designed with this method, K = 1.0. For the example frame given in Figure 5.2, the additional lateral load is based on the first-order drift ratio, Δ/L, and the total gravity load, Y_i. Thus, with $\Delta = \Delta_{1st}$,

$$\Delta_{1st}/L = 92.78/6000 = 0.0154$$

$$Y_i = 1.2(147.96 + 147.96 + 90)\ kN$$

$$Y_i = 463.10\ kN$$

$$N_i = 2.1(\Delta_{1st}/L)Y_i \geq 0.0042Y_i$$

$$N_i = 2.1(0.024)(463.10\ kN) \geq 0.0042(463.10\ kN)$$

$$N_i = 23.24 \geq 1.94$$

$$N_i = 23.24\ kN$$

23.24 kN is applied in horizontal direction.

It was previously determined in the illustration of design by second-order analysis example that the second-order drift is less than 1.5 times the first order drift. Additionally

$$\alpha P_r \leq 0.5P_y$$

$$\alpha P_r = 711\ kN$$

And for a $W14 \times 120$,

$$0.5P_y = o.5 f_y A_g$$

$$0.5P_y = o.5 \times 34500 \times 35.3 \times 25.4^2$$

$$0.5P_y = 3928540.53\ kN$$

$$\alpha P_r \leq 0.5P_y$$

Because $\Delta_{2nd} < 1.5\Delta_{1st}$ and $0.5P_y = 0.5F_yA_g$, the use of this method is permitted. The loading for this method is the same as that shown in Figure 5.2, except for the addition of a notional load of 23.24 kN coincident with the lateral load, resulting in a column moment, M_u, of 772 kNm.

This moment must be amplified by B_1 as determined from Equation C2-2 [1]. The Euler buckling load is calculated with $K_1 = 1.0$. Thus, For $P - \delta$ amplification.

$$B_1 = \frac{C_m}{1 - \alpha P_r/P_{e1}} \geq 1$$

$$C_m = 0.6 - 0.4(M_1/M_2) = 0.6$$

$$P_{e1} = \pi^2 EI/\left(KL\right)^2$$

$$P_{e1} = \pi^2 \times 2 \times 10^5 \times 1380 \times (25.4)^4/\left(1 \times 6000\right)^2$$

$$P_{e1} = 34.49 \times 10^6\ N$$

$$B_1 = \frac{0.6}{1 - \left(1 \times \frac{711 \times 10^3}{31.49 \times 10^6}\right)} \geq 1$$

$$B_1 = 0.61$$

Calculated $B_1 = 0.61 \leq 1$ hence

$$B_1 = 1$$

The axial force and associated design parameters for this method are: $P_r = 785$ kN

$K_x = Ky = 1.0$

$L_x = L_y = 6$ m

The amplified moment and associated design parameters for this method are $M_{rx} = B_1M_u = 1.0$ (772 kN-m) $= 772$ kN-m

Based on these design parameters, the axial and strong-axis available flexural strengths of the ASTM A992[21] $W14 \times 120$ are:

$P_c = \phi_c P_n$

$P_c = 5873.13 \times 10^3$ N

$M_{cx} = 1077.96$ kNm

To determine which interaction equation is applicable, the ratio of the required axial compressive strength to available axial compressive strength must be determined.

$$\frac{P_r}{P_c} = \frac{785}{5873}$$

$$\frac{P_r}{P_c} = 0.13$$

Thus, because $P_r/P_c < 0.2$, Equation H1-1b[1] of AISC 360-05 is applicable.

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0$$

$$= 0.066 + \left(\frac{772 \times 10^6}{1077.96 \times 10^6}\right)$$

Interaction ratio = 0.783

5.5.3 Design by Direct Analysis (Appendix 7)

For all frames designed with this method, $K = 1.0$.

It was previously determined in the illustration of design by second-order analysis example that the second-order drift is less than 1.5 times the first-order drift.

Thus, the notional load can be applied as a minimum lateral load, and that minimum is:

$$\Delta_{1st}/L = 92.78/6000 = 0.0154$$

$$Y_i = 1.2(147.96 + 147.96 + 90)\ kN$$

$$Y_i = 463.10\ kN$$

$$N_i = 0.002(463.10)\ kN)$$

$$N_i = 0.92\ kN$$

Because this notional load is less than the actual lateral load, it need not be applied. For a load combination that does not include a lateral load, the notional load would need to be included in the analysis.

For Column AB, using first-order analysis and B_1-B_2 amplification:

P_{nt} = 279 kN, P_{lt} = 432 kN

M_{nt} = 0 kNm, M_{lt} = 648 kNm

To determine the second-order amplification, the reduced stiffness, EI^*, must be calculated.

$$\alpha P_r = 711\ kN$$

And for a $W14 \times 120$,

$$0.5P_y = o.5 f_y A_g$$

$$0.5P_y = o.5 \times 34500 \times 35.3 \times 25.4^2$$

$$0.5P_y = 3928540.53\ kN$$

$$\alpha P_r \leq 0.5P_y$$

thus, because $0.5P_y = 0.5F_yA_g$ and $\tau_b = 1.0$.

$$EI^* = 0.8\tau_b EI$$

For P-δ amplification, since there are no moments associated with the no-translation case, there is no need to calculate B_1. For P-Δ amplification, the reduced stiffness EI^* must be used to determine the first-order drift. Because $EI^* = 0.8EI$, the first-order drift based upon EI^* is 25% larger than that calculated previously. Thus,

$$\Delta_{1st} = 93.73/0.8$$

$$\Delta_{1st} = 115.92\ mm$$

$$\Delta_{1st}/L = 0.0193\ mm$$

For moment frames, $R_M = 0.85$ and from Equation C2-6b[1] of AISC 360-05 with $\Delta_H = \Delta_{1st}$ and $\Sigma H = 72kN$,

$$\Sigma P_{e2} = R_M \Sigma H/(\Delta_{1st}/L)$$

$$\Sigma P_{e2} = 0.85(106.25)/(0.0193)$$

$$\Sigma P_{e2} = 3.167 \times 10^6\ kN$$

For design by LRFD, $\alpha = 1.0$ and ΣP_{nt} is the sum of the gravity loads. Thus,

$$B_2 = \frac{1}{1 - \frac{\alpha \sum P_{nt}}{\sum P_{e2}}} \geq 1$$

$$B_2 = \frac{1}{1 - \frac{\alpha \sum 108}{\sum 3167}} \geq 1$$

$$B_2 = 1.035$$

$EA^* = 0.8EA$, in members that contribute to lateral stability is also required in this method. However, to simplify this problem it is assumed that are no axial deformations that impact the stability of the structure. The amplified axial force (Equation 5.2) and associated design parameters for this method are:

$$P_r = P_{nt} + B_2 P_{lt}$$

$$P_r = 279 + 1.035(432)$$

$$P_r = 726.12\ kN$$

$$M_{rx} = B_1 M_{nt} + B_2 M_{lt}$$

$$M_{rx} = 1.0(0) + 1.035(648)$$

$$M_{rx} = 670.68\ kN - m$$

$$\frac{P_r}{P_c} = \frac{726.12 \times 10^3}{5873.13 \times 10^3}$$

$$\frac{P_r}{P_c} = 0.12 < 0.2$$

Thus, because $P_r/P_c < 0.2$, Equation H1-1b [1] is applicable.

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0$$

$$= 0.062 + \left(\frac{670.68 \times 10^6}{1077.96 \times 10^6}\right)$$

Interaction ratio = 0.684

5.5.4 Simplified method(AISC Basic Design Values Cards)

For the example frame given in Figure 5.2, the minimum lateral load based upon the total gravity load, Y_i is

Y_i: = 1.2(147.96 + 147.96 + 90)

Y_i = 463.10 kN

N_i = 0.002 Y$_i$

N_i = 0.002 (463.10)kN)

N_i = 0.92 kN

Because this notional load is less than the actual lateral load, it need not be applied. The 135 kN lateral load produces slightly less drift than that corresponding to the design story drift limit because the $W14 \times 120$ has I = 5161.27×10^6 mm^4 (versus the 1669.7×10^6 mm^4 required to limit drift to L/200). The actual first-order drift of the trial frame corresponds to a drift ratio of H/200 and the load ratio is:

$$1.0(463.10)/(216) = 2.14$$

Entering the table in the row for H/200, the corresponding multiplier for a load ratio of 2.24 is 1.0. Because this multiplier is less than 1.5, $\Delta_{2nd} < 1.5\Delta_{1st}$, the use of this method is permitted.
Additionally in this case, ratio of second-order drift to first-order drift is equal to or less than 1.1, K = 1.0 can be used. The amplified axial force (with the full axial force amplified by B_2) and associated design parameters for this method are:

$$P_r = 1.0P_u$$

$$P_r = 711\ kNP_u$$

The amplified moment (with the full moment amplified by B_2) and associated design parameters for this method are:

$$M_{rx} = 1.0M_u$$

$$M_{rx} = 666.14M_u$$

Based on these design parameters, the axial and strong-axis flexural available strengths of the ASTM A992[21] $W14 \times 120$s are:

$$P_c = \phi_c P_n$$

$$P_c = 5873.13 \times 10^3 N$$

$$M_{cx} = \phi_b M_{nx}$$

$$M_{cx} = 795/0.7375 \times 10^6\ Nmm$$

$$M_{cx} = 1077.96 \times 10^6\ Nmm$$

To determine which interaction equation is applicable, the ratio of the required axial compressive strength to available axial compressive strength must be determined.

$$\frac{P_r}{P_c} = \frac{711 \times 10^3}{5873.13 \times 10^3}$$

$$\frac{P_r}{P_c} = 0.12 < 0.2$$

Thus, because $P_r/P_c < 0.2$, Equation H1-1b [1] is applicable.

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0$$

$$= 0.060 + \left(\frac{666. \times 10^6}{1077.96 \times 10^6}\right)$$

$$\textbf{Interaction ratio} = \mathbf{0.678}$$

5.6 Summary

All methods produce similar designs. The result of the beam-column interaction equation for each method is:

The following conclusions can be drawn from the above examples:

Table 5.1: Comparison of Interactions Ratio

Method	**Interaction Ratio**
Second-Order (manual)	0.697
First-Order (manual)	0.78
Direct Analysis (manual)	0.684
Simplified method	0.678
STAAD Pro (Direct analysis)	0.68

Table 5.2 Comparison of Analysis Methods

	Direct analysis	**Effective length method**	**First order analysis**
Specification reference	Appendix 7 Section	C.2.2a Section	C2.2b
Limitation on use	None	$\Delta_{2nd} / \Delta_{1st} \leq 1.5$	$\Delta_{2nd} / \Delta_{1st} \leq 1.5$ $\alpha P_r \leq 0.5 P_y$
Member stiffness used in analysis	Reduced EA and EI	Nominal EA and EI	Nominal EA and EI
Analysis type	Second order analysis[1]	Second order analysis[1]	First order analysis
Column effective length	K =1	Sides sway buckling analysis[2]	K =1
Geometry of structure	Undeformed geometry of the structure in the analysis		

1. Either general second order analysis method is used or second order analysis by amplified can be used
2. K=1 is permitted for moment frame when $\Delta_{2nd} / \Delta_{1st} \leq 1.1$

- The Direct Analysis Method includes nominal geometric imperfection and stiffness reduction effects directly within the structural analysis and allows the use of K = 1.0 in calculating the in-plane column strength. The Effective Length Method, in contrast, includes the above effects indirectly within the member strength equations.

- The Effective Length and First-Order Analysis Methods have limited applicability; the Direct Analysis Method is applicable to all structures.

- While doing this study with AISC 360-05 Chapter C, Stability Analysis and Design It is found that Code requirement can only be met by checking Member Sizes and Connections using Forces from Second Order Analysis. Member Sizes which work with typical first order analysis forces did fail when designed with forces from Second Order Analysis.

- To meet code requirement, Design Members and Connections by Second Order Analysis forces. For AISC 360-05 Second Order Analysis, use direct analysis given in Appendix - 7. This is the preferred method and you can easily do it through STAAD Pro[18].

- The Direct Analysis Method (AISC 360-05 Appendix 7) is, however, the most powerful and versatile of the available methods and, as noted, it is applicable to all structures, unlike the other approaches. In new AISC 360-10 gives first preference to direct analysis method and other two methods have a second preference. The Direct Analysis Method will become the "standard" method of design for stability in near future.

Chapter 6

IS:800-2007 : Specification for Stability Design

6.1 General

This chapter explains Indian standard IS:800-2007 "General Construction In Steel - Code of Practice" provision related to stability of steel structure.

Any of the following method of structural analysis may be used to determine the design forces and moment in a member or a connection complying with the requirement of limit state of stability, strength serviceability as described in section 4 of IS:800-2007[3].

a. Elastic analysis.

b. plastic analysis.

c. advanced analysis.

d. Dynamic analysis.

The procedure to perform all these analysis are mentioned in section 4 of IS:800-2007[3] in detail.

6.2 Assumption in analysis

Notional Horizontal Loads

To analyze a frame subjected to gravity loads, considering the sway stability of the frame, notional horizontal forces should be applied. These notional horizontal forces account for practical imperfections and should be taken at each level as being equal to 0.5% of factored dead load plus vertical imposed loads applied at that level. The notional load should not be applied along with other lateral loads such as wind and earthquake loads in the analysis.

The notional forces should be applied on the whole structure, in both orthogonal directions, in one direction at a time, at roof and all floor levels or their equivalent, They should be taken as acting simultaneously with factored gravity loads.

a. The notional force should not be, applied when considering overturning or overall instability;

b. The notional force should not be combined with other horizontal (lateral) loads;

The notional force should not be combined with temperature effects; and The notional force should not be taken to contribute to the net shear on the foundation.

6.3 Methods of structural analysis as per IS:800-2007

(a) Elastic analysis:

it is based on the assumption that no fiber of the member has yielded for the design load and stress is linearly proportional to strain. The analysis may be in two stages.

Stage 1: First order analysis; is based on load acting on deformed geometry of the structure redistribution of 15% peak moment is permitted by code.

Stage 2: second order analysis: it is based on deformed shape of the structure. IS:800-2007 permits use of amplification factors instead of second order analysis based on limitation.

(b) Plastic analysis:

In this method it is assumed that when every fiber at a section reaches yield stress a plastic hinge is formed. After hinge is formed, it is assumed that the member rotates freely at the plastic hinge without resisting any additional moment. Its resistance constant (M_p) is called first order plastic analysis. Code permits second order inelastic analysis by any of the following methods.

a. Distributed plasticity method.

b. Elastic-plastic method.

c. Modified plastic hinge method.

(c) Second order analysis

In a second-order elastic analysis, the members shall be assumed to remain elastic, and changes in frame geometry under the design load and changes in the effective stiffness of the members due to axial forces shall be accounted for. In a frame where the elastic buckling load factor of the frame as determined in accordance with 4.6 of IS:800-2007 is greater than 5, the changes in the effective stiffness of the members due to axial forces may be neglected.

The design bending moment under factored load shall be taken as the maximum bending moment in the length of the member. It shall be determined either:

a. directly from the second-order analysis; or

b. approximately, if the member is divided into a sufficient number of elements, as the greatest of the element end bending moments; or

c. by amplifying the calculated design bending moment, taken as the maximum bending moment along the length of a member as obtained by superposition of the simple beam bending moments determined by the analysis.

For a member with zero axial force or a member subject to axial tension, the factored design bending moment shall be calculated as the moment obtained from second order analysis without any amplification.
For a member with a design axial compressive force as determined from the analysis, the factored design bending moment shall be calculated as follows

$$M = \delta_b M_m$$

moment amplification factor for a braced member determined in accordance with Section 9 of IS:800 2007[3].

(d) Advance analysis:
Where the moment amplification factor C_Y, C_Z, calculated is greater than 1.4, a second-order elastic analysis in accordance with Annex B shall be carried out.

For frame members of compact section with full lateral restraints, an advance structural analysis may be carried out, provided the analysis can be shown to accurately model the actual behaviour of that class of frames. The analysis shall take into account the following:

a. Relevant material properties.

b. Residual stress.

c. Geometric imperfections.

d. Reduction in stiffness due to axial compressions.

e. Second order effects.

f. Section strength and ductility.

g. Erection procedure.

h. Interaction with foundation.

For design it shall be sufficient to satisfy the section capacity requirements of IS:800-2007[3] of Section 8 (Design of member subjected to bending) for the members subjected to bending, of Section 7 (Design of compression members) for axial members, of Section 9(Member subjected to combined forces) for combined forces and of Section 10(Connection) for connections.

Effect of moment magnification given in Section 9[3] (Member subjected to combined forces), instability given in Section 7 (Design of compression members) and lateral buckling given in Section 8 (Design of member subjected to bending) need not be considered while designing the member, since advanced analysis methods directly consider these.

An advanced structural analysis for earthquake loads shall recognize that the design basis earthquake loads calculated in accordance with IS:1893 is assumed to correspond to the load at which the first significant plastic hinge forms in the structure.

6.4 Design philosophy of IS:800-2007

"Limit States" are the various conditions in which a structure would be considered to have failed to fulfil the purpose for which it was built. In general two limit states are considered at the design stage and these are limit state of strength and limit state of serviceability. "Limit State of Strength" are: loss of equilibrium of the structure and loss of stability of the structure. "Serviceability Limit State" refers to the limits on acceptable performance of the structure. The earlier version of code IS:800-1984[5] is based on allowable stress design, in allowable stress design the basic form of calculations took the form of verifying that the stresses caused by the characteristic loads must be less than an "allowable stress", which was a fraction of the yield stress. Thus the allowable stress may be defined in terms of a "factor of safety" which represented a margin for overload and other unknown factors which could be tolerated by the structure.

In general, each member in a structure is checked for a number of different combinations of loading. The value of factor of safety in most cases is taken to be around 1.67. Many loads vary with time and these should be allowed for. It is unnecessarily severe to consider the effects of all loads acting simultaneously with their full design value, while maintaining the same factor of safety or safety factor. Using the same factor of safety or safety factor when loads act in combination would result in uneconomic designs.

6.5 Strength of Beam-Columns as per IS:800-2007

The behaviour of beam-columns is fairly complex, particularly at the ultimate stage and hence exact evaluation of the strength would require fairly complex analysis. However, for design purposes, simplified equations are available, using which it is possible to obtain the strength of members, conservatively.

(a) Section Strength:

Plastic and Compact Sections: The design of members subjected to combined axial force (tension or compression) and bending moment, the following should be satisfied.

$$\left(\frac{M_y}{M_{ndy}}\right)^{\alpha_1} + \left(\frac{M_z}{M_{ndz}}\right)^{\alpha_2} \leq 1.0$$

Conservatively, the following equation may be used under combined axial force and bending moment

$$\frac{P}{P_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0$$

where, M_y, M_z = factored applied moments about the minor and major axis of the cross section, respectively.

M_{ndy}, M_{ndz} = design reduced flexural strength under combined axial force and the respective uniaxial moment acting alone.

P = factored applied axial force.

P_d = design strength in compression due to yielding given by

$$P_d = A_g f_y / \gamma_{mo}$$

γ_{mo}= Partial factor of safety in yielding.

M_{dy}, M_{dz} = design strength under corresponding moment acting alone.

A_g = gross area of the cross section.

α_1,α_2 = constants.

Semi-compact section

$$\frac{P}{P_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0$$

(b) Overall Member Strength: Members subjected to combined axial compression and moment shall be checked for overall buckling failure as given below:

$$\frac{P}{P_{dy}} + k_y \frac{C_{my} M_y}{M_{dy}} + k_{LT} \frac{M_z}{M_{dz}} \leq 1.0$$
$$\frac{P}{P_{dz}} + 0.6\, k_y \frac{C_{my}\, M_y}{M_{dy}} + k_z \frac{C_{mz}\, M_z}{M_{dz}} \leq 1.0$$

where, C_{my}, C_{mz} = Equivalent uniform moment factor.

P = applied axial tension or compression under factored load.

M_y, M_z = maximum factored applied bending moments about y and z-axis of the member, respectively.

P_{dy}, P_{dz} = design strength under axial tension or compression as governed by buckling about minor (y) and major (z) axis respectively.

M_{dy}, M_{dz} = design bending strength about y (minor) or z (major) axis of the cross section.

$$K_y = 1 + (\lambda_y - 0.2)n_y \leq 1 + 0.8n_y$$

$$K_z = 1 + (\lambda_z - 0.2)n_y \leq 1 + 0.8n_z$$

$$K_{LT} = 1 - \frac{0.1\lambda_{LT}n_y}{(C_{mLT} - 0.25)} \geq 1 - \frac{0.1n_y}{(C_{mLT} - 0.25)}$$

where, n_y, ${}_nz$ = ratio of actual applied axial force to the design axial strength for buckling about the y and z axis, respectively.

C_{mLT} = Equivalent uniform moment factor for lateral torsional buckling.

λ_y, λ_z = Non dimensional slenderness ratio about the minor and major axis respectively.

6.6 Strength of Beam-Columns as per IS:800-1984

Interactive formula with a factor of safety n as 1.67 and incorporates a reduction factor C_m to consider the end condition and side sway of the column in frames, which should be multiplied by the amplified bending stress ratio.

$$\frac{\sigma_{ac,cal}}{\sigma_{ac}} + \frac{C_{mx}\sigma_{bcx,cal}}{\left\{1 - \frac{\sigma_{ac,cal}}{0.6f_{ccx}}\right\}\sigma_{bcx}} + \frac{C_{my}\sigma_{bcy,cal}}{\left\{1 - \frac{\sigma_{ac,cal}}{0.6f_{ccy}}\right\}\sigma_{bcy}} \leq 1$$

$\sigma_{bcx,cal}$ = calculated bending compressive stress due to the bending moment about major axis.

σ_{bcx} = permissible bending compressive stress about major axis taking into account lateral instability.

$\sigma_{bcy,cal}$ = calculated bending compressive stress due to the bending moment about minor axis.

σ_{bcx} = permissible bending compressive stress about minor axis taking into account lateral instability.

C_m = a coefficient called reduction factor whose value is established by relative size and direction of the column end moment and never more than one.

a. Side sway not prevented, i.e. no bracing against sidesway buckling is provided, C_m =0.85

b. For braced column, side sways is prevented and not subjected to transverse load between support in the plane of bending, C_m = 0.6-0.4$\beta \geq 0.4$
β = ratio of smaller to larger moment at the ends of the members.

6.7 An Example: Three storey one bay frame

A three storey one bay moment frame as shown in Figure 6.1 has been analyzed and design the column AB using IS:800-2007 and IS:800-1984.
Strength of beam column has been evaluated from three method.

- First order analysis using moment amplification as per IS:800-2007.
- Second order analysis as per IS:800-2007 (Annex B1).
- Advanced analysis as per IS:800-2007 (Annex B2).
- Using IS:800 1984.

(a) First order analysis using moment amplification as per IS:800-2007

First order elastic analysis Using load combination = 1.2 Dead Load (Gravity load) + 1.2 Wind Load (Horizontal Load) carried out.
Using Staad Pro v8i[18] bending moment and Axial force are calculated:
Moment at Joint B = 499 kNm
Axial Force in column AB = 603 kN

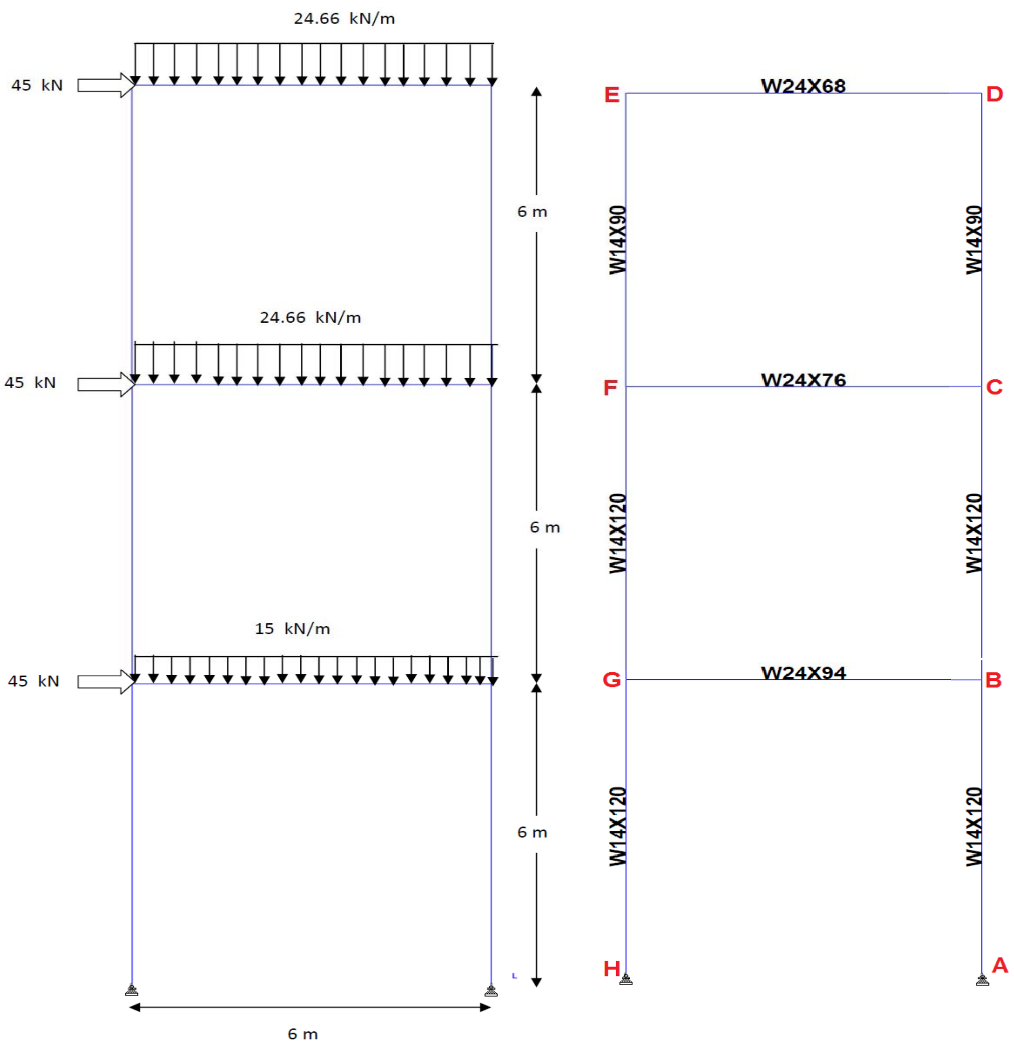


Figure 6.1: One-Bay Moment Resisting Frame

The detail manual calculation using excel program is prepared for the problem given in Appendix B.

$$\frac{P}{P_{dy}} + k_y \frac{C_{my} M_y}{M_{dy}} + k_{LT} \frac{M_z}{M_{dz}} = 0.64$$

$$\frac{P}{P_{dz}} + 0.6\, k_y \frac{C_{my}\, M_y}{M_{dy}} + k_z \frac{C_{mz}\, M_z}{M_{dz}} = 0.53$$

(Detail calculation refer Appendix B)

(b) Advanced analysis using Annex B of IS:800-2007

In Annex B of IS:800-2007, It is specified that the Advance analysis is an option to take care at analysis stage all non linearity i.e. Relevant material properties, Residual stress, Geometric imperfections, Reduction in stiffness due to axial compressions and Second order effects. If advance analysis is carried out for the structure than Effect of moment magnification given in Section 9, instability given in Section 7 and lateral buckling given in Section 8 need not be considered while designing the member, since advanced analysis methods directly consider these.

IS:800-2007 dose not provide detailed requirements of "Advance analysis". It dose not clearly specific acceptable method/procedure to take in to account various non-linearities.

AISC 360 -05 clearly specifies that the direct analysis consider all these nonlinearity at analysis stage by applying The notional load, reducing flexural and axial stiffness. Direct analysis is type of advance analysis. Hence to carry out Advance analysis method according to AISC 360-05 is used for analysis and for design IS:800-2007 is used.

Here the example of one bay three storey frame is analyzed using Direct analysis method according to AISC 360-05 and design as per IS:800-2007 of Column AB. Eliminating effective length factor K (K = 1) and moment magnification factor (K_y, K_z = 1)

Carried out first order elastic analysis using load combination = 1.2 Dead Load (Gravity load) + 1.2 Wind Load (Horizontal Load). Load case
Moment at Joint B = 527 kNm
Axial Force in column AB = 617 kN
For advance analysis factor are taken in design is as followed:
Effective length factor = 1

K_y= 1

K_z = 1

X_{lt}=1

using above constant interaction ratio is calculated for detail calculation is given in Annex B

$$\frac{P}{P_{dy}} + k_y \frac{C_{my} M_y}{M_{dy}} + k_{LT} \frac{M_z}{M_{dz}} = 0.68$$
$$\frac{P}{P_{dz}} + 0.6\, k_y \frac{C_{my}\, M_y}{M_{dy}} + k_z \frac{C_{mz}\, M_z}{M_{dz}} = 0.43$$

Calculation of Interaction Ratio as per IS:800-2007 for combined axial and bending = 0.68

(Detail calculation refer Appendix B)

(c) Design as per IS:800 1984

First order elastic analysis is used to calculate the member forces Load combination is taken as per IS:800 1984

Load case = 1.0 Dead Load(Gravity load) + 1.0 Wind Load(Horizontal Load)

Moment at Joint B = 416 kNm

Axial Force in column AB = 502 kN

Interactions ratio:

$$\frac{\sigma_{ac,cal}}{\sigma_{ac}} + \frac{C_{mx}\sigma_{bcx,cal}}{\left\{1 - \frac{\sigma_{ac,cal}}{0.6 f_{ccx}}\right\}\sigma_{bcx}} + \frac{C_{my}\sigma_{bcy,cal}}{\left\{1 - \frac{\sigma_{ac,cal}}{0.6 f_{cc}y}\right\}\sigma_{bcy}} = 0.701$$

(Detail calculation refer Appendix B)

6.8 Summary

Method	Interaction Ratio
First order analysis using moment amplification - IS:800-2007	0.64 0.53
Advanced analysis using Annex B - IS:800-2007	0.68 0.53
As per IS:800-1984	0.70

In this chapter carried out a design of column AB as shown Figure 6.1 as per IS:800-2007 and IS:800-1984 and presented Interaction ratio of Column AB. few important points of this study is summarized in this section the list of the same as follows:

- IS:800-2007 it is clearly specified that if advance analysis is consider Effect of (nonlinear effect i.e. material properties , Residual stress ,Geometric imperfections , Reduction in stiffness due to axial compressions, Second order effects, Section strength and ductility and Erection procedure) moment magnification (Section 9 of IS:800-2007), instability (Section 7 of IS:800-2007) and lateral buckling (Section 8 of IS:800-2007) at analysis time so need not be considered while designing the member. But IS:800-2007 dose not list the factor in design which can be set as 1(i.e. it's effect in design is not required).

- Carried out analysis and design based on IS:800-2007 and it is seen that the increment of 5% in stress ratio while using advance analysis compared to second order analysis. While using IS:800 1984 the stress ratio is increased by 9% which is high as compared IS:800-2007 methods.

Chapter 7

Case study I

Stability Analysis and Design of Piperack Structure According to AISC Specification

7.1 General

Piperack is the main arterial system of a process plant. It consists of an overhead structure supporting the process pipes which are connecting equipment, and the lines entering and leaving a unit. Utility lines, supplying steam, water, air, gas to process equipment and relief valve headers, instrument cables and electrical cables are supported on piperack. Piperack is usually constructed of steel or concrete frames or combination of steel and concrete frame. Figure 7.1 shows a typical piperack structure.

In this chapter, general arrangement of piperack structure, design criteria for loads and load combinations are briefly introduced.

Figure 7.1: Steel Piperack

The Piperack structure selected for this case study is taken from an executed project. The structural arrangement and loads due to pipes are taken as basic input. Wind load is calculated manually.

The piperack 3-D frames are modeled in STAAD Pro.[18]. The analysis is carried out according to various methods prescribed by AISC 360-05[1](explained in detail in chapter 5). To demonstrate application of each method, all three methods namely (1) First order analysis (2) Second order analysis (3) Direct analysis, are applied to the problem one by one. For sake of comparison amongst results derived from different methods, one typical transverse frame is selected (Frame on Axis 18). Based on results, a summary for application of AISC:360-05 stability provisions to similar plant structures is presented.

7.2 Design Criteria and Specifications

- In this Piperack structure, all frames in transverse direction are rigidly connected and frames in longitudinal direction are braced.

- All columns have pinned connection with the foundation pedestal.
- All secondary beams are designed as a simply supported and thus have shear connections.
- All members are designed as per AISC 360-05 by using LRFD method.

7.3 Computer Model

The structure is analysed by computer program STAAD Pro.V8i(20.07.07.19)[18]. The general system of the computer model is with global axis system:
X = Horizontal axis in computer model along West - East direction.
Y = Vertical axis in computer model (positive upward direction).
Z = Horizontal axis in computer model along North - South direction.
For the present case study, the stability of the main structural framing system is of the interest. Hence small platforms, hangers, cantilever beams and brackets are not modeled, but the reactions of the same are transferred to the main frame to get equivalent effect. Also, stairs are not modeled but their reactions have been transferred to its supporting members.
For detail structural layout refer Appendix A.

7.4 Type of Loading

Various loads acting on piperack structure are briefly explained in this section. All primary loads cases and load combination and considered for analysis are listed in Appendix B.

7.4.1 Dead Loads

a. Appropriate density (7850 kg/m^3) are defined for the structural members from which their self weights are considered.

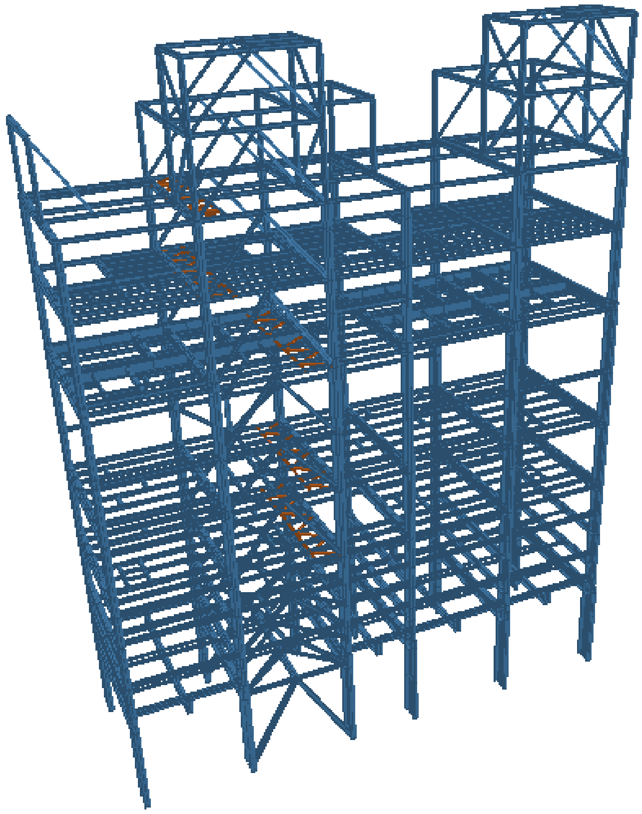


Figure 7.2: 3-D STAAD model view of structure

b. Electrical and Instrumentation cable loads along with the tray/duct supporting systems are considered.

c. Weight of walkways and platforms along with grating (6 kN/m^2 considering 35 mm thick grating) are considered.

d. The dead load of ladder and hand rail is taken as a 0.25 kN/m.

7.4.2 Imposed Loads

Imposed loads are taken as 5 kN/m^2. This includes the weight of all movable loads including personnel, tools, miscellaneous equipments, movable partition, cranes, hoists, parts of dismantled equipment and stored material.

7.4.3 Piping and equipments loads

All the types of Pipe Loading (Vertical and Horizontal) shall be considered as per given in Appendix A. Weight of the equipment is given in Appendix A.

7.4.4 Additional Reserve Loads

Additional reserve loads in vertical as well horizontal direction are considered in the calculation to take care of loads caused by utility lines, instrument and control devices etc. and variation in piping loads.
Additional vertical downward reserve load of 50 kN is considered at the top of each column and Additional horizontal force of 10 kN is considered at every level, refer appendix A.

7.4.5 Wind Loads

Wind load is calculated as per ASCE-7- 2005[22]. For detail calculation refer ApendixA

7.4.6 Thermal Loads

Thermal Loads are caused by change in temperature. Such forces shall include those caused by vessel or piping expansion or contraction. Thermal forces act at piping restrained supports. Piping thermal loads are marked on Appendix A.

7.5 Stability analysis of piperack structure

Stability analysis is carried out using AISC 360-05 provision. Comparison has been made for all three methods first order analysis, second order analysis and direct analysis method for piperack frames highlighted in Figure 7.3.

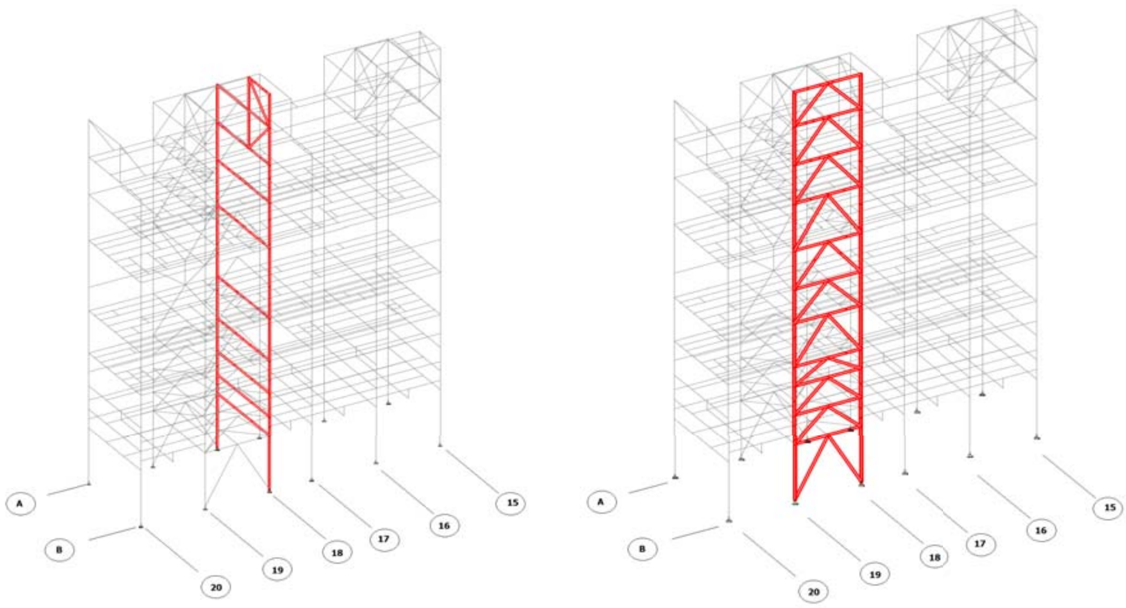


Figure 7.3: Piperack structure (1). Axis 18 moment direction (2). Raw B braced direction

7.6 Results and Discussion

Comparison has been shown in terms of bending moment. Shear force, deflection and interaction ratio. In present study for comparison axis 18 and row B has been taken as shown in Figure 7.3 are highlighted.

For comparison in moment direction the governing load combination 319 is taken: Load Combination 319 = 1.2 Dead Load (DL) + 1.0 Live Load (LL) + 1.2 Operating weight of piping/equipment (D_{LOP}) - 1.6 Wind load in direction(WL_Z) - 1.2 Pipe thermal load in X direction (TL_X) - 1.2 Pipe thermal load in Z direction (TL_Z) + 1.2 Reserve load Vertical direction (R_V) - 1.2 Reserve load horizontal in X direction (R_{HE}) - 1.2 Reserve load horizontal in Z direction (R_{HN}).

For comparison in bracing direction the governing load Combination 316 is taken: Load Combination 316 = 1.2 Dead Load (DL) + 1.0 Live Load (LL) + 1.2 Operating weight of piping/equipment (D_{LOP}) + 1.6 Wind load in X direction (WL_X) +

1.2 Pipe thermal load in X direction (TL_X) + 1.2 Pipe thermal load in Z direction (TL_Z) + 1.2 Reserve load Vertical direction (R_V) + 1.2 Reserve load horizontal in X direction (R_{HE}) + 1.2 Reserve load horizontal in Z direction (R_{HN}).

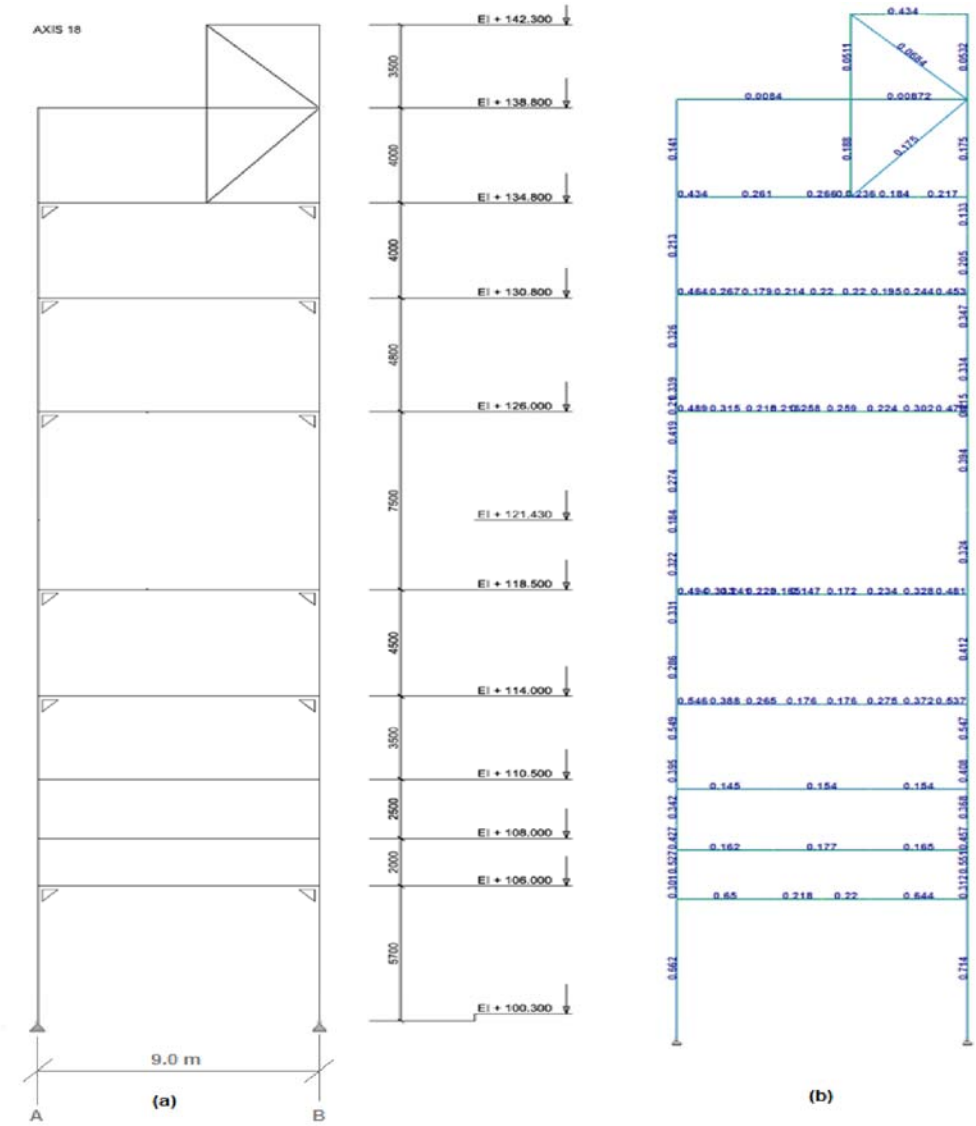


Figure 7.4: Piperack structure(a) Axis 18 (b) Interaction ratio of Axis 18 - Direct analysis

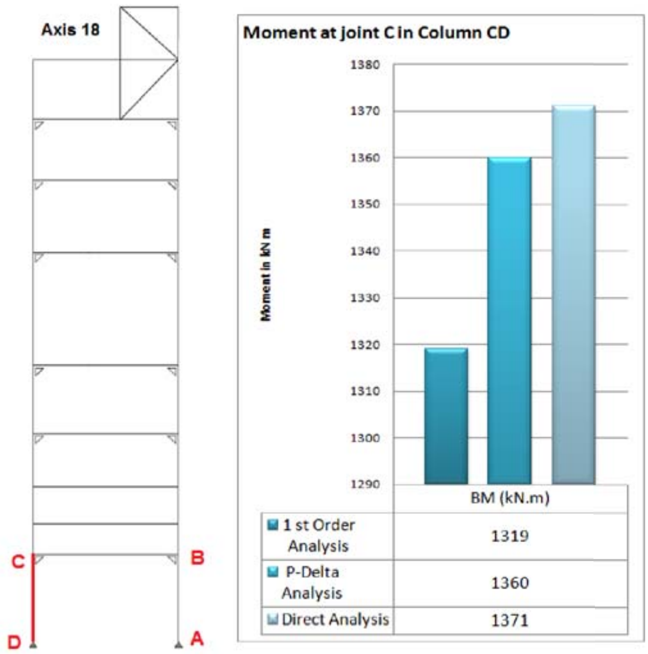


Figure 7.5: Axis 18, Comparison of B.M. in column CD at joint C

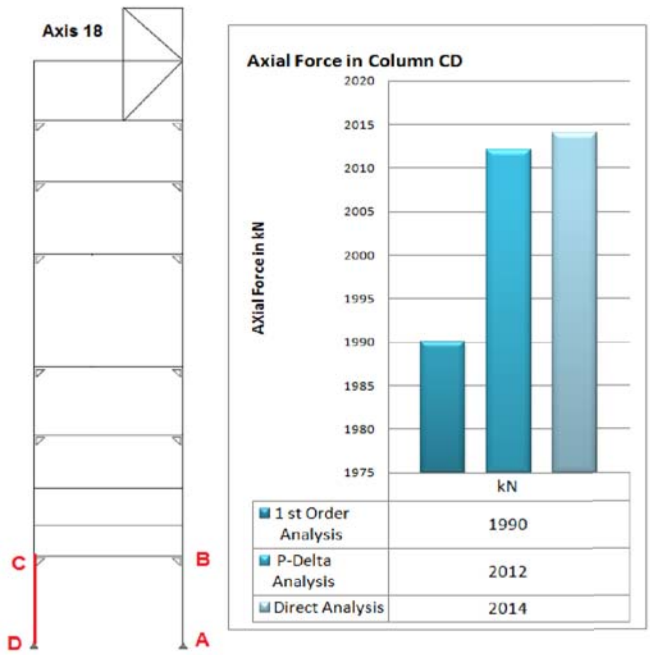


Figure 7.6: Axis 18, Comparison of Axial of force in column CD

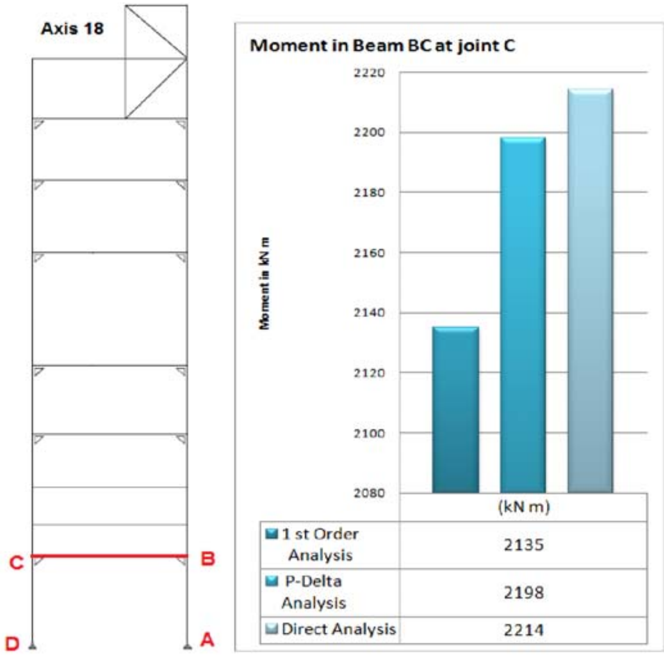


Figure 7.7: Axis 18, Comparison of B.M. in beam BC at joint C

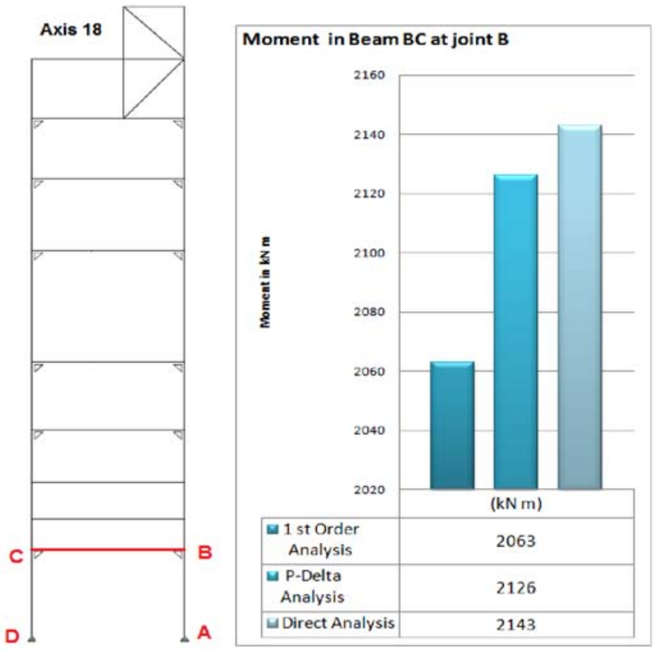


Figure 7.8: Axis 18, Comparison of B.M. in Beam BC at joint B

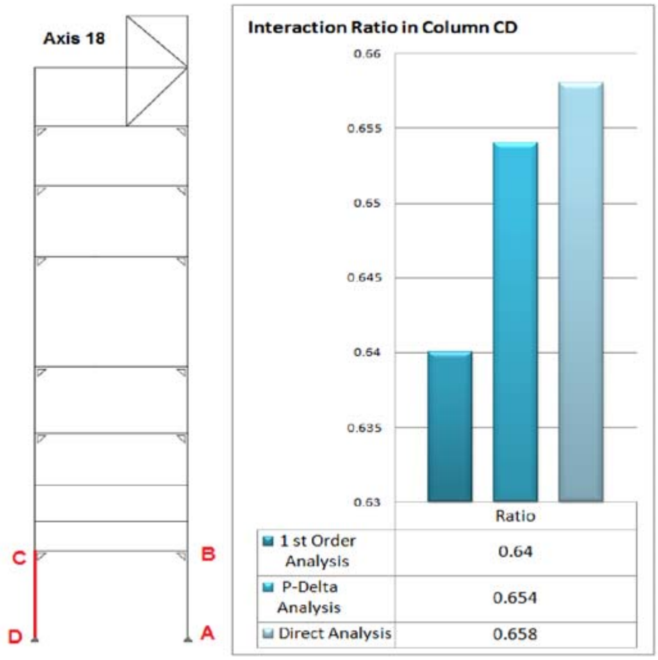


Figure 7.9: Axis 18, Comparison of interaction ratio in column CD

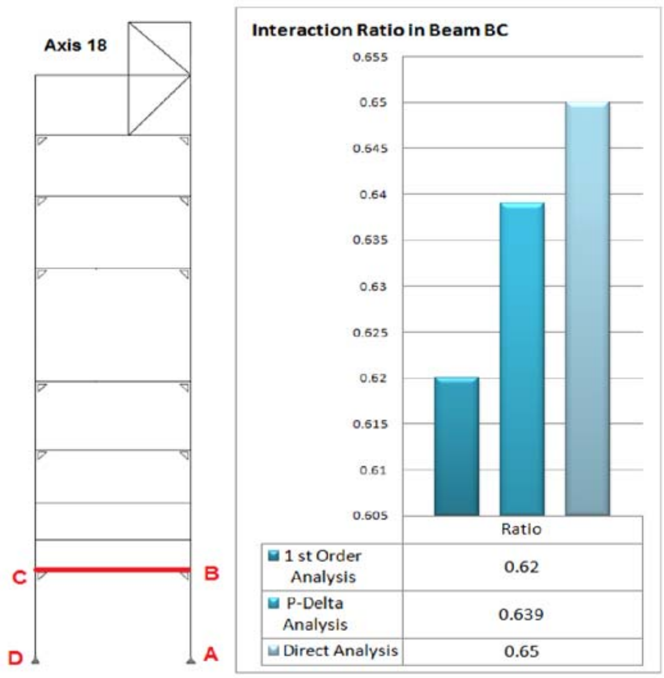


Figure 7.10: Axis 18, Comparison of interaction ratio in Beam BC

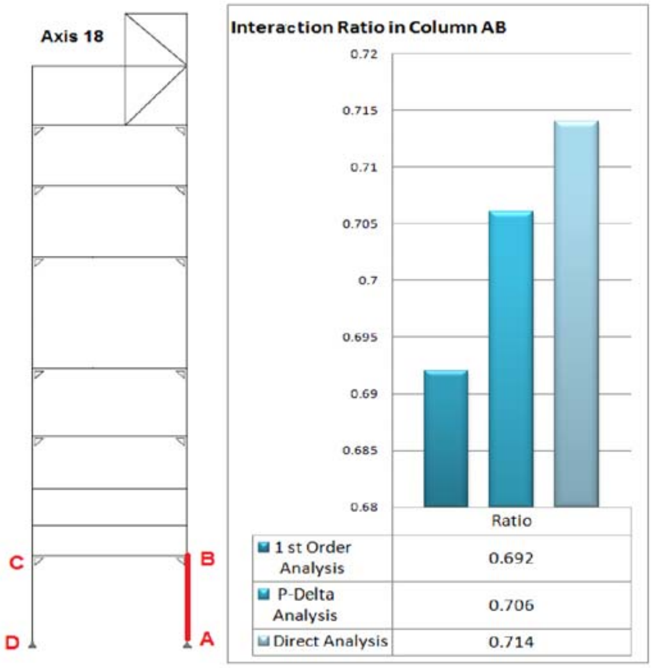


Figure 7.11: Axis 18, Comparison of interaction ratio in column AB

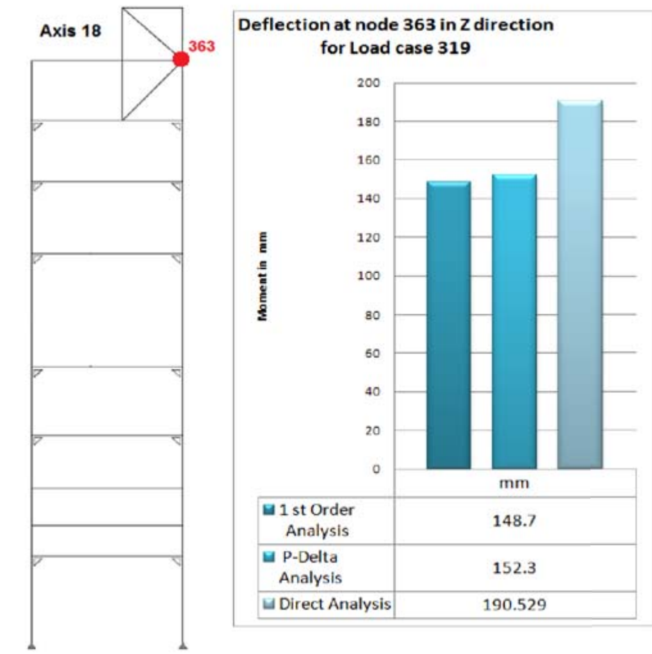


Figure 7.12: Axis 18, Comparison of Deflection at node 363

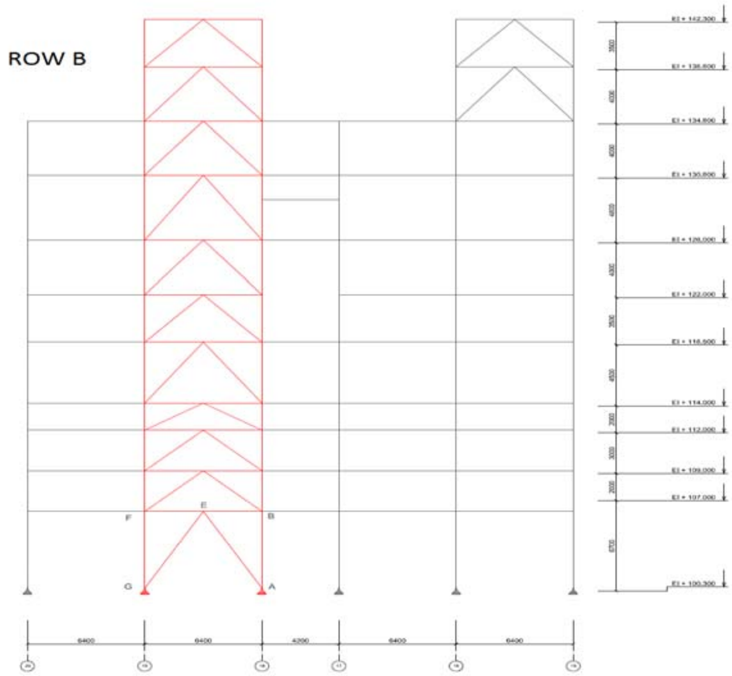


Figure 7.13: Row B of piperack structure

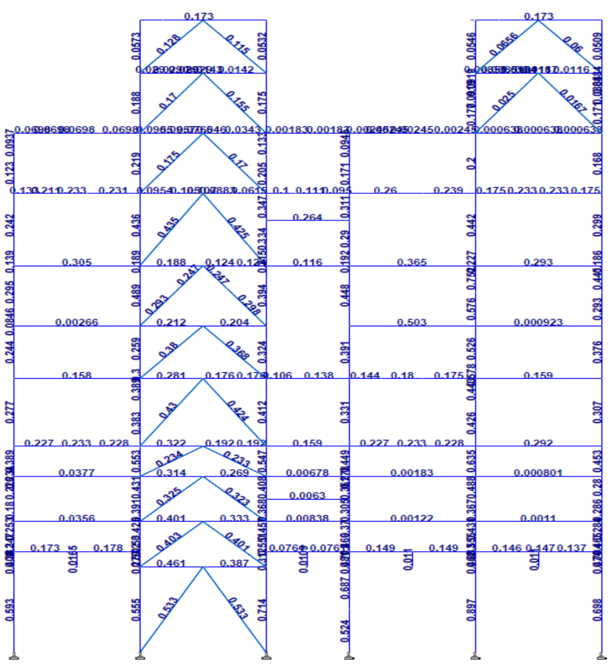


Figure 7.14: Interaction Ratio of Row B - Direct analysis

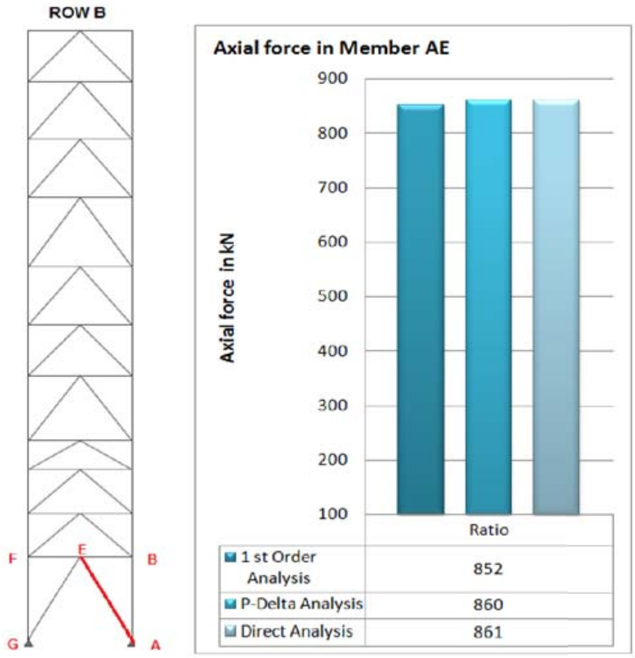


Figure 7.15: Row B, Comparison of axial force in Member AE

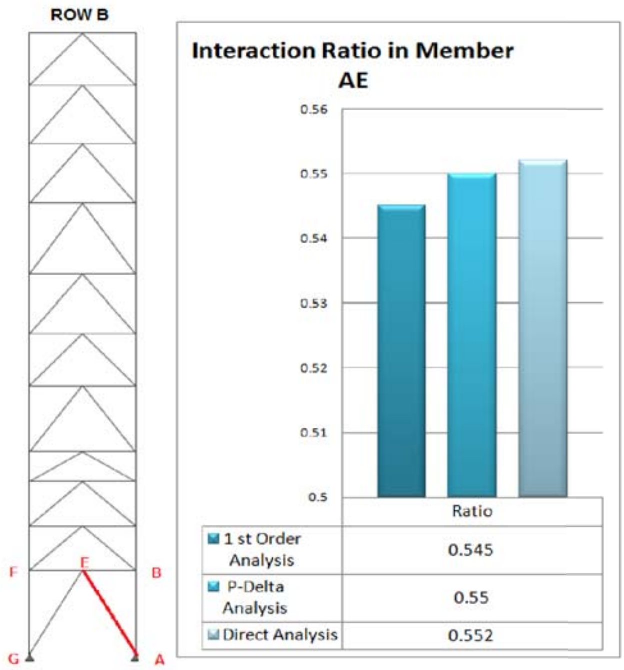


Figure 7.16: Row B, Comparison of interaction ratio in Member AE

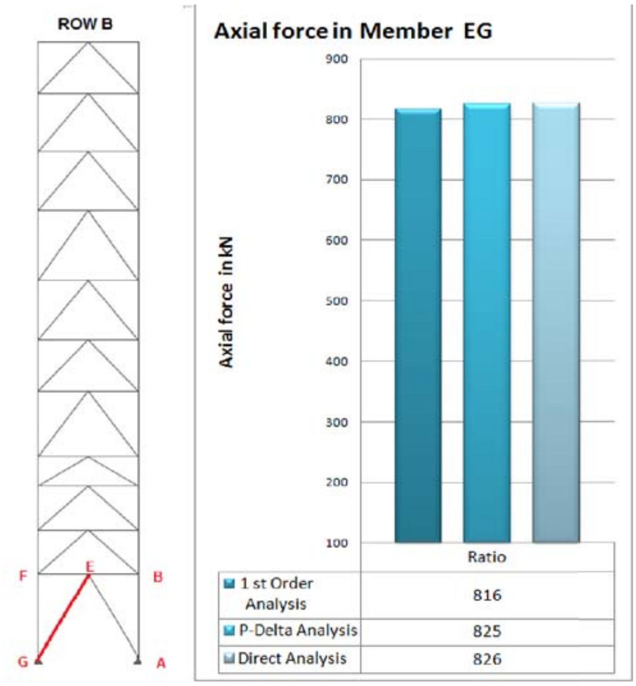


Figure 7.17: Row B, Comparison of axial force in Member EG

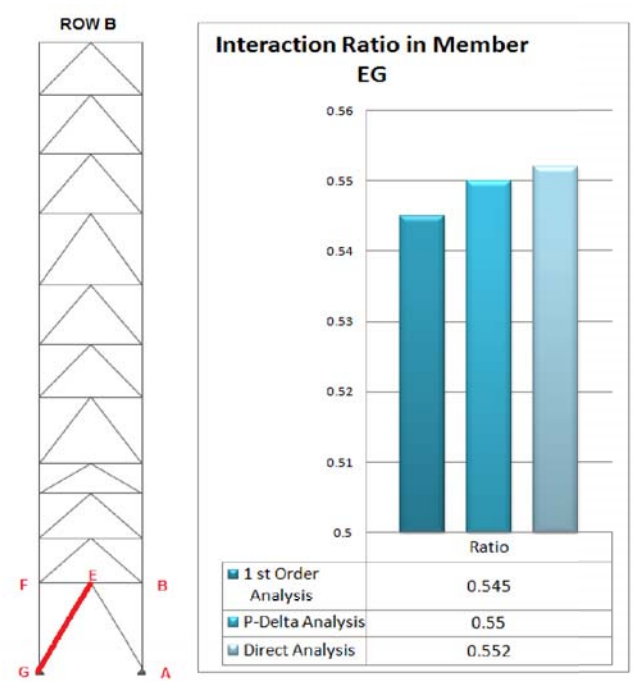


Figure 7.18: Row B, Comparison of interaction ratio in Member EG

Table 7.1: Axis 18, Comparison of B.M. in column CD at joint B

Analysis Type	**BM (kN.m)**	**% Change**
1 st Order Analysis	1319	0.00
P-Delta Analysis	1360	+3.11
Direct Analysis	1371	+3.94

Table 7.2: Axis 18, Comparison of Axial force in column CD

Analysis Type	**Axial force (kN)**	**% Change**
1 st Order Analysis	1990	0.00
P-Delta Analysis	2012	+1.11
Direct Analysis	2014	+1.21

Table 7.3: Axis 18, Comparison of B.M. in beam BC at joint C

Analysis Type	**B.M.(kNm)**	**% Change**
1 st Order Analysis	2135	0.00
P-Delta Analysis	2198	+2.95
Direct Analysis	2214	+3.70

Table 7.4: Axis 18, Comparison of B.M. in Beam BC at joint B

Analysis Type	**B.M. (kNm)**	**% Change**
1 st Order Analysis	2063	0.00
P-Delta Analysis	2116	+3.05
Direct Analysis	2143	+3.88

Table 7.5: Axis 18, Comparison of interaction ratio in column CD

Analysis Type	**Interaction Ratio**	**% Change**
1 st Order Analysis	0.64	0.00
P-Delta Analysis	0.65	+3.05
Direct Analysis	0.658	+3.88

Table 7.6: Axis 18, Comparison of interaction ratio in Beam BC

Analysis Type	**Interaction Ratio**	**% Change**
1 st Order Analysis	0.62	0.00
P-Delta Analysis	0.639	+3.05
Direct Analysis	0.65	+4.84

Table 7.7: Axis 18, Comparison of interaction of ratio in Column AB

Analysis Type	**Interaction Ratio**	**% Change**
1 st Order Analysis	0.692	0.00
P-Delta Analysis	0.706	+2.2
Direct Analysis	0.714	+3.18

Table 7.8: Axis 18, Comparison of Deflection at node 363

Analysis Type	**Deflection mm**	**% Change**
1 st Order Analysis	148.7	0.00
P-Delta Analysis	152.3	+2.4
Direct Analysis	190.52	+28.1

Table 7.9: Row B, Comparison of axial force in Member AE

Analysis Type	**Axial Force(kN)**	**% Change**
1 st Order Analysis	852	0.00
P-Delta Analysis	860	+0.94
Direct Analysis	861	+1.06

Table 7.10: Row B, Comparison of interaction ratio in Member AE

Analysis Type	**Interaction ratio**	**% Change**
1 st Order Analysis	0.545	0.00
P-Delta Analysis	0.550	0.92
Direct Analysis	0.552	1.28

Table 7.11: Row B, Comparison of axial force in Member EG

Analysis Type	**Axial force kN**	**% Change**
1 st Order Analysis	816	0.00
P-Delta Analysis	825	+1.10
Direct Analysis	826	+1.23

Table 7.12: Row B, Comparison of interaction ratio in Member EG

Analysis Type	**interaction ratio**	**% Change**
1 st Order Analysis	0.551	0.00
P-Delta Analysis	0.554	0.54
Direct Analysis	0.553	0.36

7.7 summary

Three approaches has been studied i.e. first order analysis, Second order explicit analysis by P-delta analysis and direct analysis as per AISC 360-05[1] for stability analysis for piperack structure. The result has been shown for Axis 18 moment frame (transverse direction) and row B (Braced direction).

According to AISC 360-05, K factor is considered as:
K = 1 for first order analysis.
K = according to alignment chart method for second order analysis.
K = 1 for direct analysis.

In Moment direction(transverse direction): Comparing with P-delta analysis with direct analysis, direct analysis will increase the moment by 0.8% to 1% and interaction ratio will also increase by 0.6% to 0.7%.

In braced direction (Longitudinal direction): comparing all the three method there is no change in interaction because in braced direction the K factor is approximately 1, see Figure 7.18.

In case of deflection direct analysis deflection increases by 28% (Figure 7.12) compare to first order analysis but as point out in AISC 360[1] commentary, Appendix 7 does not apply to serviceability condition of excessive deflection. Direct analysis results is increase of the connection forces and also increase in the base plate and bolt size.

As per new AISC 360-2010 specification now the direct analysis method is a standard method given in chapter C[23] and it is applicable to all type of structure, the P-delta analysis and first order analysis has a limited applicability.

Chapter 8

Case study II

Stability Analysis and Design of Piperack Structure According to AISC Specification

8.1 General

Analysis and design is carried out of piperack structure using IS:800-2007 and IS:800-1984 provision. Comparison has been made for all three methods first order moment amplification analysis, Advance analysis as per IS:800-2007 and as per IS:1984 for piperack frames highlighted in Figure 7.3.

Basic load and load combination are taken as presented in Appendix A. For load combination table 4 of IS:800-2007 is followed and presented in Appendix A and for IS:800-1984 it is taken as per 3.4.2.1 of IS:800-1984 and presented in Appendix A.

Comparison of following three methods has been discussed in this chapter.

1. First order analysis with moment amplification as per IS 800:2007

2. Advance analysis as per IS:800-2007 (Annex B)

3.as per IS 800:1984.

Input data (i.e. Load calculation, Basic load case and Load combination) for piperack structure is taken as specified in Appendix A. For IS:800-2007 load combination has been prepared as specified in table 18 and for IS:800-1984 as specified in section 3.4.2.1 of respective code.

8.2 Results and Discussion

Comparison has been made for two directions (a) moment direction axis 18 (b) braced direction row B as highlighted in Figure 7.3 has been shown in terms of bending moment, shear force, deflection and interaction ratio. All results has been shown for load case:

Detail load case and Load combination for Piperack structure are given in Appendix A. In this chapter results are shown for following load combination:

IS:800-2007 load combination:

Load combination 311 = 1.5 Dead Load $((DL))$ + 1.5 Operating weight of piping or equipment (D_{LOP}) - 1.5 Wind load in X direction (WL_X) - 1.5 Pipe thermal load in Z direction (TL_Z) - 1.5 Pipe thermal load in X direction (TL_X) + 1.5 Reserve load Vertical direction (R_V) - 1.5 Reserve load horizontal in X direction(R_{HE}) - 1.5 Reserve load horizontal in Z direction (R_{HN})

IS:800-1984 load combination:

Load combination 311 = 1.0 Dead Load (DL) + 1.0 Operating weight of piping/equipment(D_{LOP}) - 1.0 Wind load in X direction(WL_X) - 1.0 Pipe thermal load in Z direction (TL_Z) - 1.0 Pipe thermal load in X direction (TL_X) + 1.0 Reserve load Vertical direction

(R_V) - 1.0 Reserve load horizontal in X direction(R_{HE}) - 1.0 Reserve load horizontal in Z direction (R_{HN})

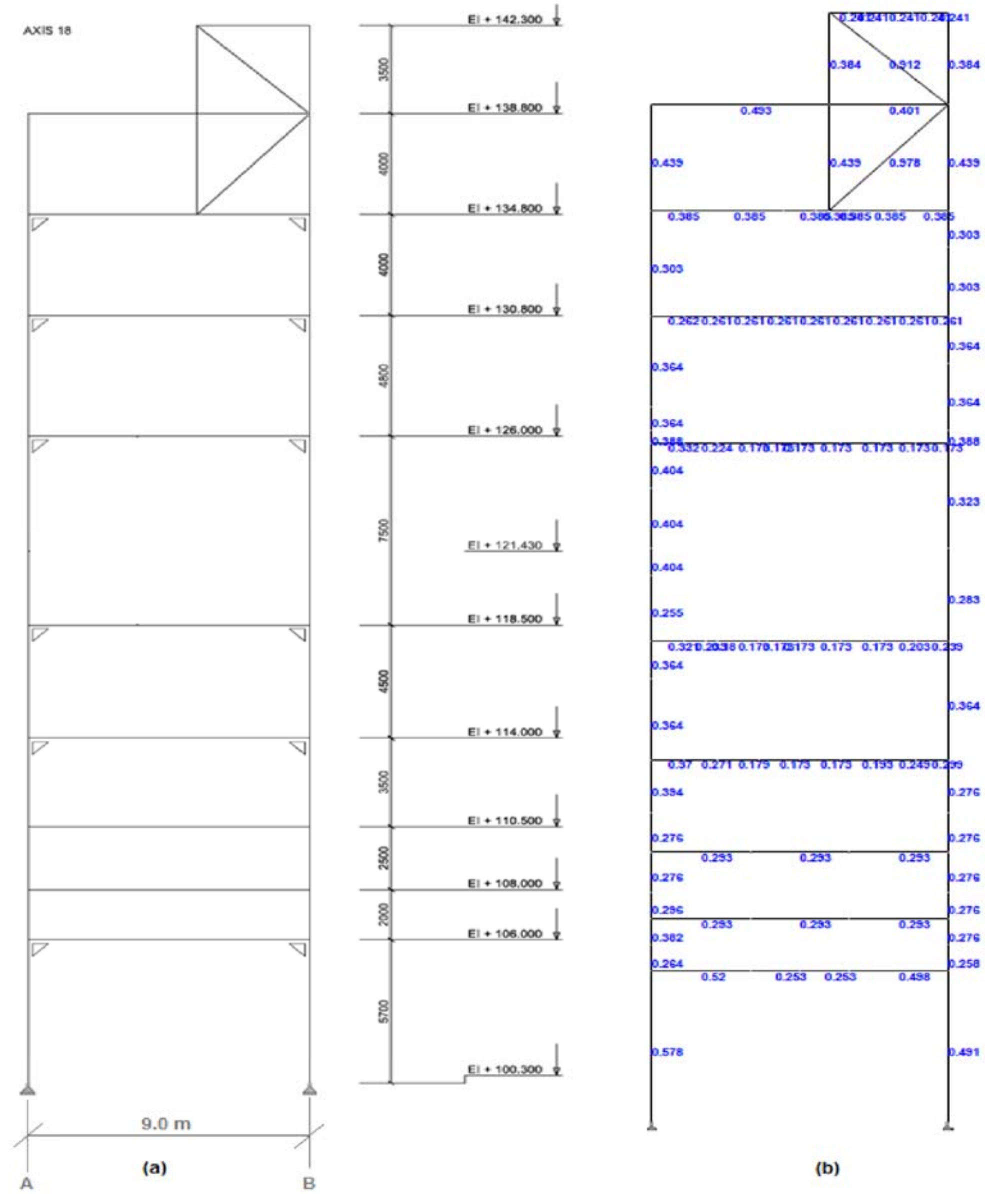


Figure 8.1: Piperack structure(a) Axis 18 (b) interaction ratio of Axis 18 - IS:800-2007

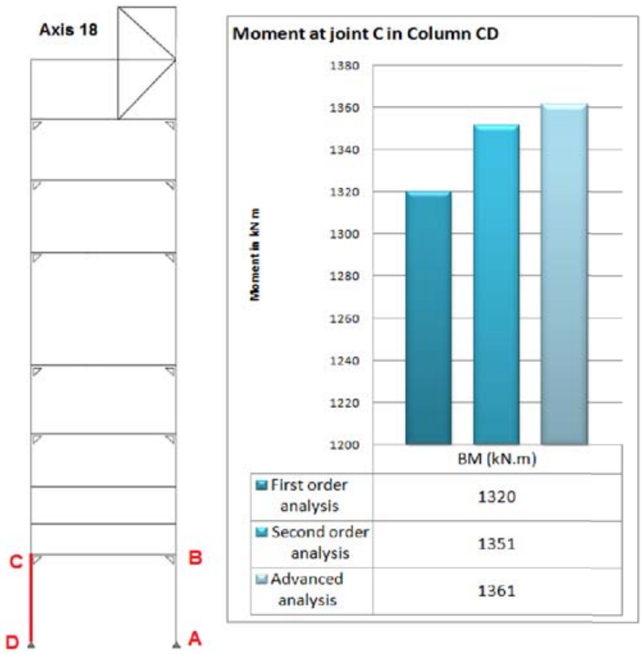


Figure 8.2: Axis 18, Comparison of B.M. in column CD at joint C

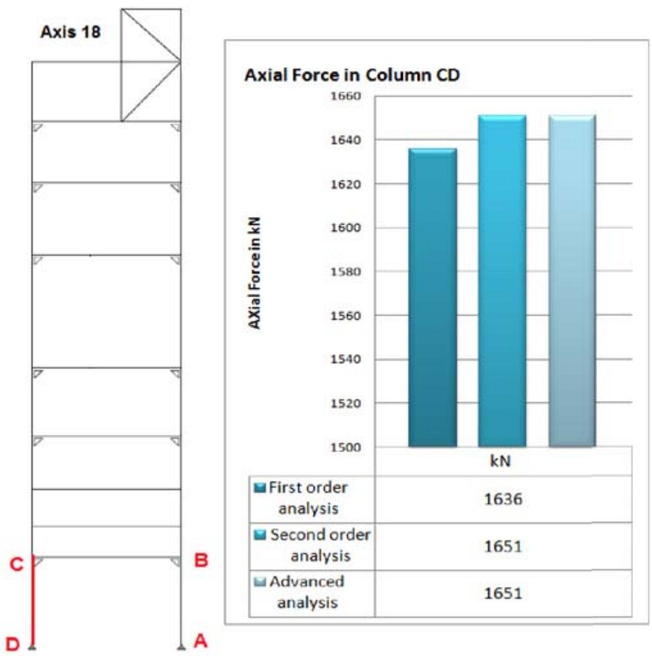


Figure 8.3: Axis 18, Comparison of Axial force in column CD

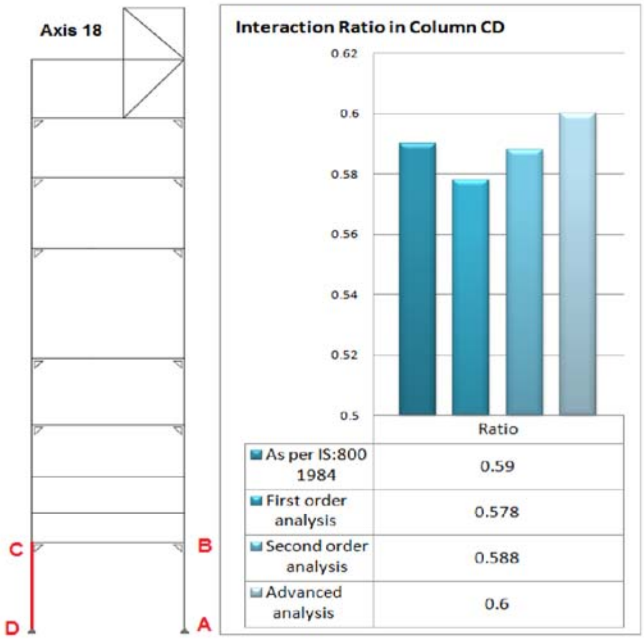


Figure 8.4: Axis 18, Comparison of Interaction ratio in column CD

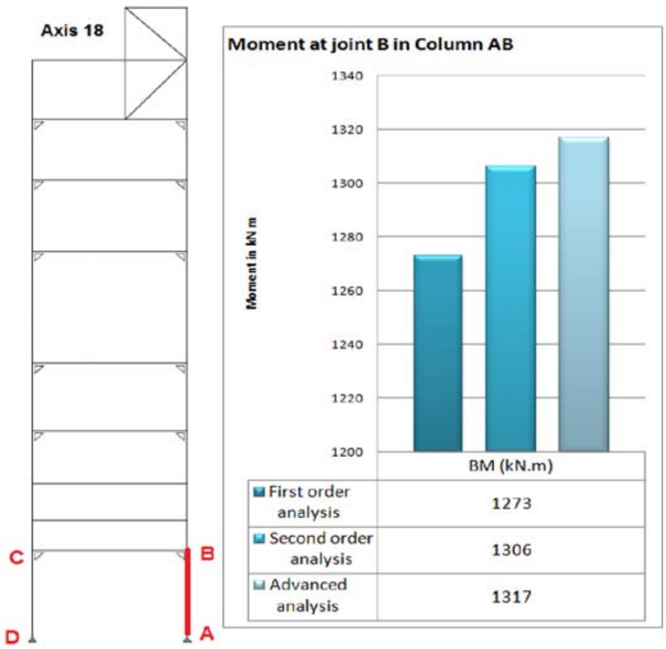


Figure 8.5: Axis 18, Comparison of B.M. in Column AB at joint B

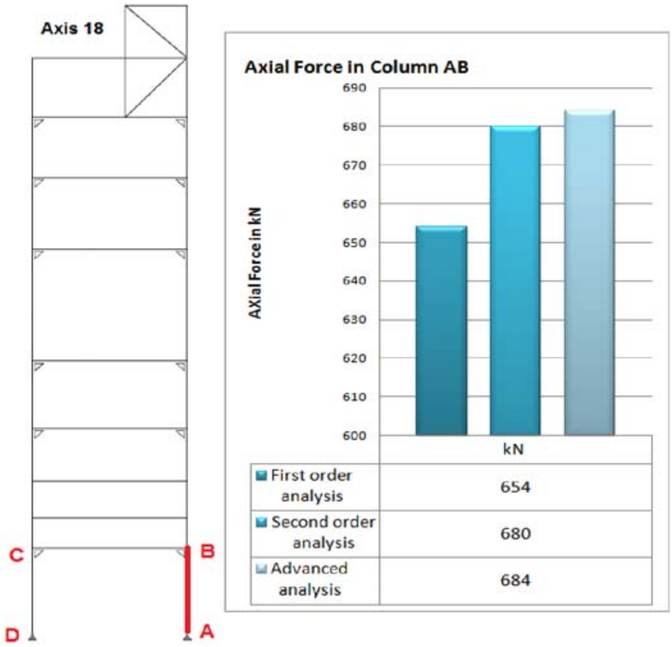


Figure 8.6: Axis 18, Comparison of axial force in column AB

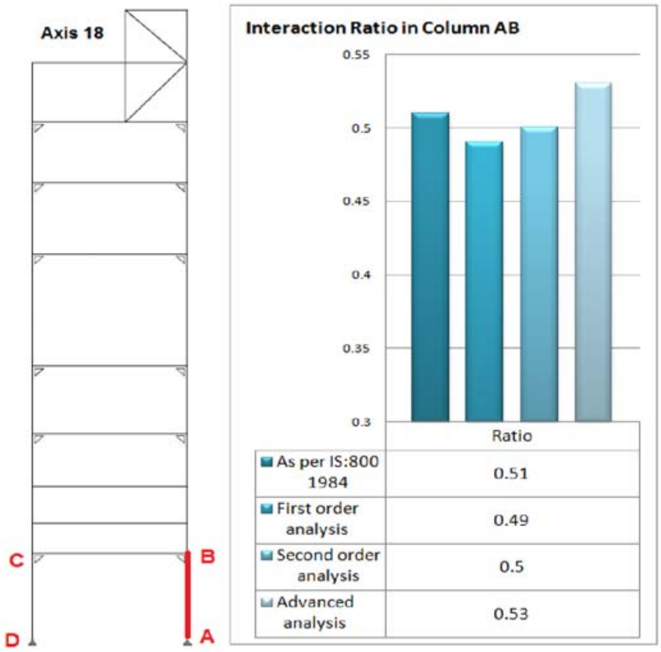


Figure 8.7: Axis 18, Comparison of interaction ratio in column AB

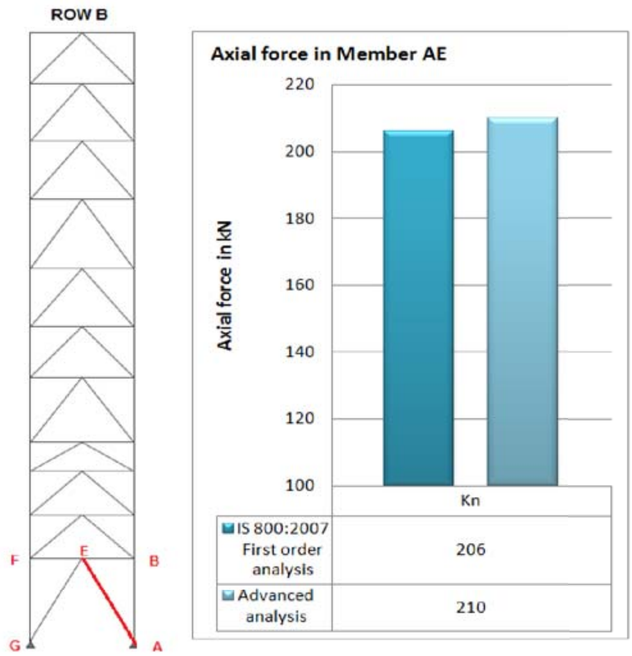


Figure 8.8: Row B, Comparison of axial force in Member AE

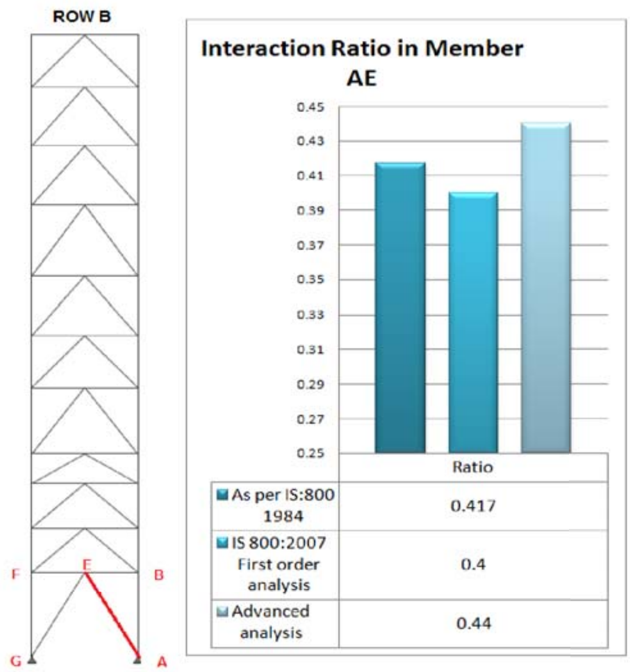


Figure 8.9: Row B, Comparison of interaction ratio in Member AE

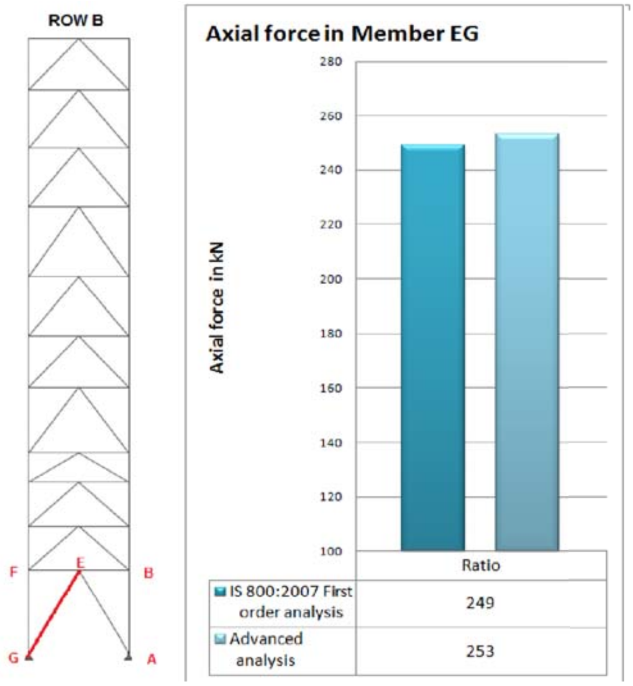


Figure 8.10: Row B, Comparison of axial force in Member EG

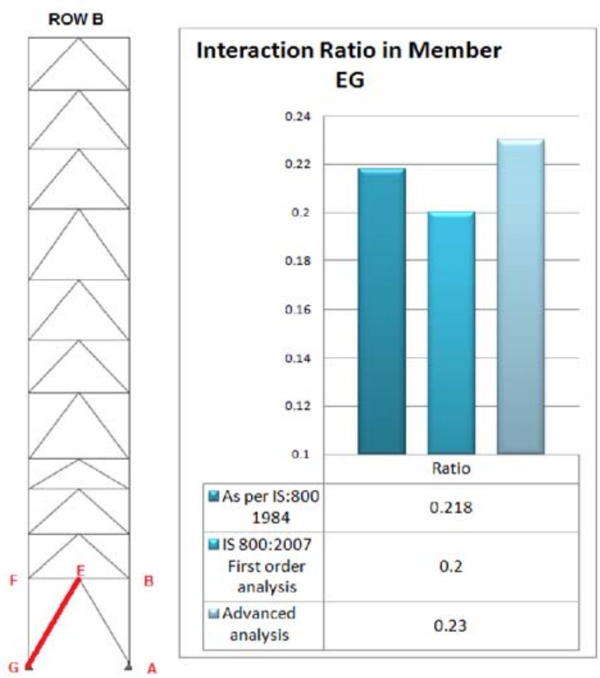
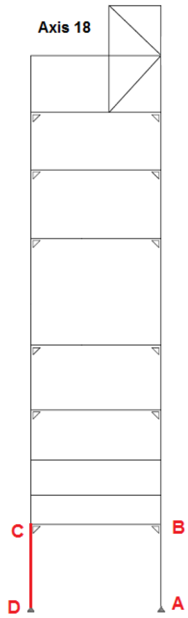
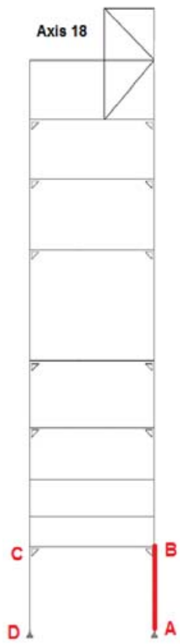


Figure 8.11: Row B, Comparison of interaction ratio in Member EG



Method	Member	Moment kN m		Axial Load kN	Factor		Interaction
First order using moment amplification	Column CD	M_c	1320 kNm	1636 kN	K_y	1.211	0.578
					K_z	1.02	
		M_d	0 kNm		X_{lt}	0.77	
					Effective length factor kz	2.1	
Second order using moment amplification (B1)		M_c	1351 kNm	1651kN	K_y	1.21	0.588
					K_z	1.02	
		M_d	0 kNm		X_{lt}	0.77	
					Effective length factor kz	2.1	
Advance analysis (B2)		M_c	1361 kNm	1651kN	K_y	1	0.6
					K_z	1	
		M_d	0 kNm		X_{lt}	1	
					Effective length factor kz	1	
As per IS:800 1984		M_c	880 kNm	1090 kN	-		0.59
		M_d	0 kN m				

Figure 8.12: Row B, Comparison of design parameter Member CD



Method	Member	Moment kN m		Axial Load kN	Factor		Interaction
First order using moment amplification	Column AB	M_c	1273kNm	654kN	K_y	1.09	0.491
					K_z	1.01	
		M_d	0 kNm		X_{lt}	0.77	
					Effective length factor kz	2.1	
Second order using moment amplification (B1)		M_c	1306 kNm	680 kN	K_y	1.09	0.5
					K_z	1.01	
		M_d	0 kNm		X_{lt}	0.77	
					Effective length factor kz	2.1	
Advance analysis (B2)		M_c	1317 kNm	684 kN	K_y	1	0.54
					K_z	1	
		M_d	0 kNm		X_{lt}	1	
					Effective length factor kz	1	
As per IS:800 1984		M_c	880 kNm	436 kN	-		0.51
		M_d	0 kN m				

Figure 8.13: Row B, Comparison of design parameter in Member AB

8.3 Summary

- The New edition of Indian standard IS:800-2007[3] presents various methods of analysis of steel structure with regard to stability. There are three approaches permitted :
  (1) First order analysis and moment amplification during design, this has limited application,
  (2) Second order elastic analysis,
  (3) Advanced structural analysis.
  - In this piperack structure, first-order elastic analysis with moment amplification is applicable because K_y and K_z is less than 1.4. In present study carried out first order analysis for piperack and results are shown in terms of bending moment shear force and interaction ratio.
  - The second order analysis with accordance with Annex B1[3] is carried out with option a as specified in IS:800 2007.
  - The advance analysis with accordance with Annex B2[3] is carried out the effect of moment amplification, instability and lateral buckling as per section 9,7 and 8 respectively are ignored.
- Because there no clear guideline in IS:800-2007[3] for advance analysis option and advance analysis have same assumption as direct analysis so for this study direct analysis is used to carry out the advance analysis. As shown in Figure 8.12 and 8.13 the factors which were considered at analysis stage are eliminated in advance analysis because it is considered in analysis stage. Comparing the interaction ratio for column CD, it is increased by maximum 1% in case of advance analysis refer Figure 8.12 and 8.13. This increment is reduced with increases in the elevation of column members (i.e. upper element).
- Factor calculated for first order amplified is neglected in Advance analysis is major advantage of this analysis. First order moment amplification method has

limitation (K_y, $K_z \leq 1.4$) whereas Advance analysis is a versatile method it is applicable to all structure. It is difficult or time consuming process calculate K_y and K_z where the number of member in the structure is large. i.e. Piperack structure.

- A notable observation on IS:800-2007[3] code is the need of further explanation required for implementation of each of these methods.

Chapter 9

Summary and Conclusions

9.1 Summary

Stability loss under compressive load is usually termed structural or geometrical instability and is commonly known as buckling. Instability is a condition wherein a compression member loses the ability to resist increasing loads and exhibits instead a decrease in load carrying capacity. In other word, instability occurs at the maximum point on the load deflection curve.

To determine the realistic failure load of an actual member it is necessary to take initial imperfection into account and to consider the entire nonlinear load deflection curve of the member. Numerous nonlinearity are present in the members due to existence of geometrical imperfections, material imperfections, residual stresses etc. The non linearity effect is either to be considered at analysis time or use amplification the factor at design time which takes care of moment amplification due to second order effect. The factor K is another mathematical adjustment to reduce the capacity of ideal column to take into account the practical imperfections.

Numerous approaches have been proposed for evaluating the K factor. But assumption made for simple approaches do not justify it's use for real structure and more "realistic" approaches result in to complex and tedious calculations. AISC:360-05 proposed a new versatile method called direct analysis method which takes care of the all nonlinear effect at analysis stage and allow K = 1

Direct analysis involves reduction of stiffness matrix term "EI" and "EA" during analysis and use full values for design. STAAD Pro package has introduced a special command to implement this and other requirement of direct analysis method. STAAD command is validated by solving one bay frame from paper published by AISC " A Comparison of Frame Stability Analysis Methods in ANSI/AISC 360-05 " and matching it's results affirmatively with STAAD Pro result[refer appendix-C].

To study the AISC stability analysis method a plane frame is solved and to demonstrate it's implementation and impact on plant structure, a pipe rack steel structure has been taken as case study. The case study structure is analyzed and designed by each method i.e. First order analysis method, second order analysis method, direct analysis method and simplified method. Results are compared in terms of structural deflection and interactions ratio, , moment and axial force of the critical members. Based on the case study-I, conclusion listed in section 9.2 are derived.

Section-4 of IS:800-2007 specifies the different analysis types and assumption behind them. Three approaches of IS:800-2007 are presented in the study: The first order analysis using moment amplification, second order analysis and advance analysis. To study the IS:800-2007 stability analysis method a plane frame is solved and to demonstrate it's implementation and impact on plant structure, same pipe rack steel structure has been taken as case study. The case study structure is analyzed and designed by each method i.e. first order moment amplification method, second order analysis, advance analysis method and using IS:800-1984 criteria. Results are compared in terms of structural deflection and interactions ratio, moment and axial force of the critical members. Based on the case study-II conclusion listed in section 9.2 are derived.

There has been substantial research work done in field of structural stability in recent past and on that basis the steel design codes/specifications have been updated to consider more refined and precise methods. Newly introduced methods named as direct analysis and advance analysis by AISC and IS code respectively are example of such modern methods. Implementation of such methods needs understanding of fundamental concept of stability.

9.2 Conclusions

- In the example frame solved in chapter 3 the first order elastic analysis and second order analysis are carried for 18 m tall moment frame. Due to second order P-delta effect the deflection at top storey is increased by 3% and bottom story moment is increases by 3% to 4% compared to first order analysis. Also it has been noticed that P-delta effect at bottom story is high as compared to top storey.
- By evaluating effective length for one bay frame in example frame of chapter 4, it is found that the Liu's methods and Lemessurier's Method gives higher value as compare to AISC alignment chart method, IS:800-2007 Method and IS:800-1984 Method. Comparing the IS:800-2007 and IS:800-1984 the IS:800-2007 introduce a new factor called connection factor which is depends on the amount of load on column and connection type on that joint. In new IS:800 2007, the equation given to calculate the effective length equation is based on graph which is same as presented in old code of IS:800-1984.
- **From Case Study I:**
  - A typical moment frame (Axis 18) and braced frame (Raw B) has been

taken to illustrate results of study.

  * In moment frame, the bending moment obtained from direct analysis is 3% and 1% higher than first order analysis and second order analysis respectively.
  * In moment frame, the axial force for columns obtained from direct analysis is 1.2% and 0.5% higher than first order analysis and second order analysis respectively.
  * In moment frame, the interaction ratio obtained from direct analysis is 3% and 1% higher than first order analysis and second order analysis respectively.
  * In braced frame there is a marginal difference of axial force and interaction ratio obtained from the first order analysis, second order analysis and direct analysis.

- Bottom storey columns are more affected due to second order analysis and negligible effect found on bracing member due to second order analysis.
- Structure has more effect of second order analysis because of P-Δ (Structure effect) and there is a negligible second order effect due to P-δ (Member effect).
- Comparing all the three methods, it is apparent that there is a marginal difference in axial force and interaction ratio in bracing member.
- In case of deflection, it is increased by 28% by direct analysis but as point out in AISC:360-05 commentary on Appendix 7; it does not apply to serviceability condition of excessive deflection.

• **From Case Study II:**

  - A typical moment frame (Axis 18) and braced frame (Raw B) has been taken to illustrate results of study.

  * In moment frame, the axial force obtained from direct analysis is 1.2% and 0.5% higher than first order analysis and second order analysis respectively.
  * o In moment frame, the bending moment obtained from advance analysis is 2% and 3% higher than second order analysis and first order analysis respectively.
  * In moment frame, the axial force in columns obtained from the advance analysis is 1% higher than second order analysis and first order analysis.
  * In moment frame, the effective length factor of column, Ky and Kz for first order analysis and second order analysis are 1.2 and 1.09 respectively where in case of advance analysis Ky and Kz is taken as 1.0.
  * In braced frame there is a marginal difference in axial force and interaction ratio obtained from all the methods.

- The IS:800-2007 has proposed a three analysis methods for design of structure. In the present study all three methods of analysis: First order analysis, second order analysis and advance analysis method are used. It is observed that for tall structures where P-Δ effect is expected to be significant (code prescribes K_y, K_z limit of 1.4 to identify second order effect); it is advisable to use second order analysis. Advance analysis approach suggested by Code is still in primitive stage. For implementation of this approach on actual project it is required that code provides more clarity and guidelines about how different non-linearity are to be incorporated in analysis stage (as clearly defined by AISC:360-2005 in case of Direct Analysis approach) and special command is required by the software package, which can take care of the different stiffness parameters during analysis and design stage internally. In present work, problem was faced due to lack of such command with STAAD Pro and hence design for critical members had to be carried out manually.

- Moment amplification factor C_y, C_z of section 4.4.2 of IS:800-2007 should be considered as K_y , K_z.

9.3 Future scope of work

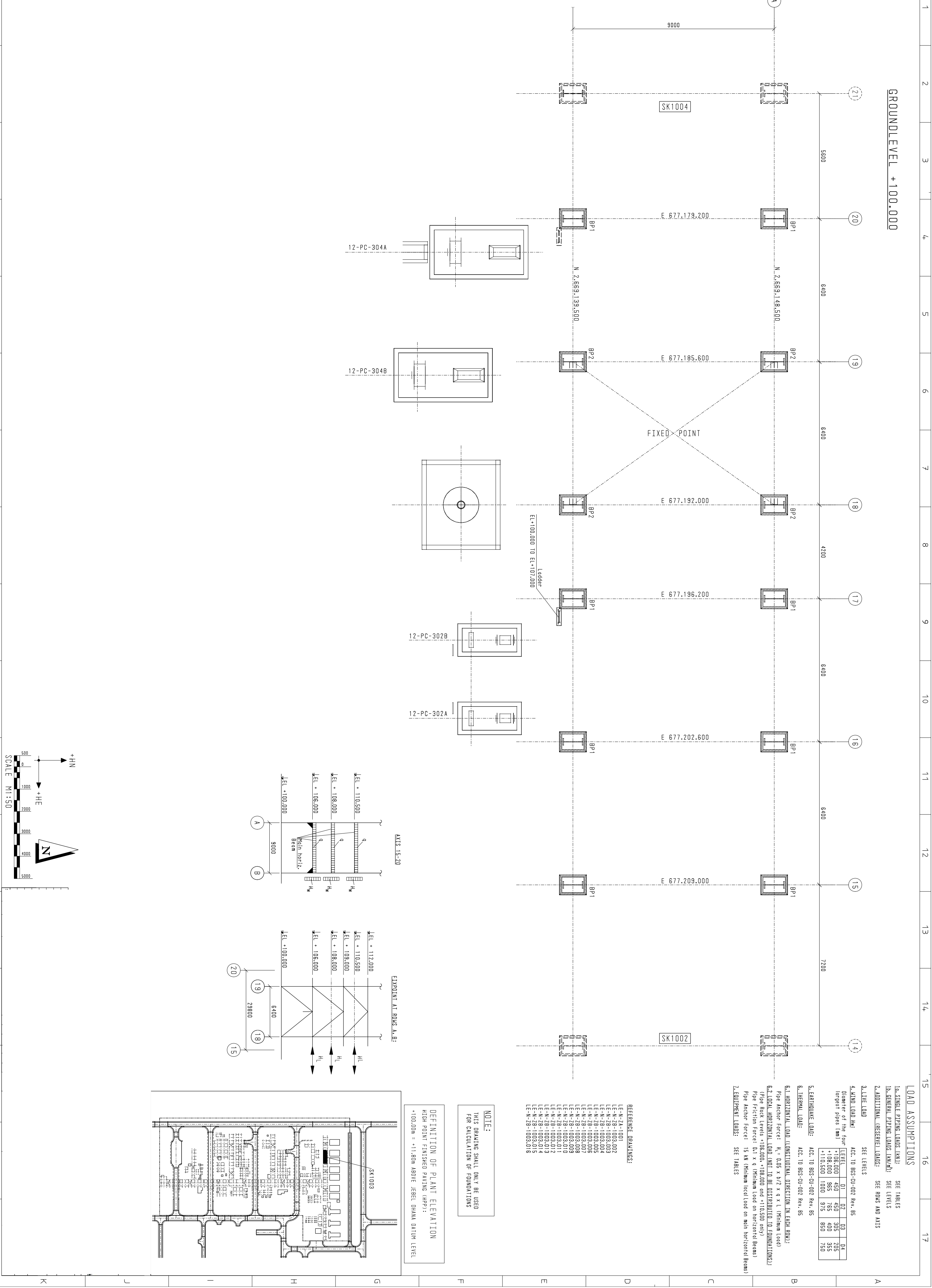
- The study in this report is limited to stability analysis of steel structure using AISC:360-05 and IS:800-2007 analysis methods. The present study can be extended to include the following aspects.
- Evaluate the K factor using buckling analysis of sway frame.
- Stability analysis of other than piperack structure by performing all the three analysis methods of AISC:360-05 and IS:800 2007 analysis procedures.
- Carry out the stability analysis of steel structure using the new AISC:360-10 and investigate the difference between 360-05 and new 360-10 provisions related to analysis.
- Explore the Advance analysis option in detail as per IS:800 2007 also carry the stability analysis using plastic analysis, frame buckling analysis described in section 4 of IS:800-2007.
- Dedicated computer software command is required can be developed for to carry out the stability analysis using Advance analysis option of IS:800-2007.

Appendix A

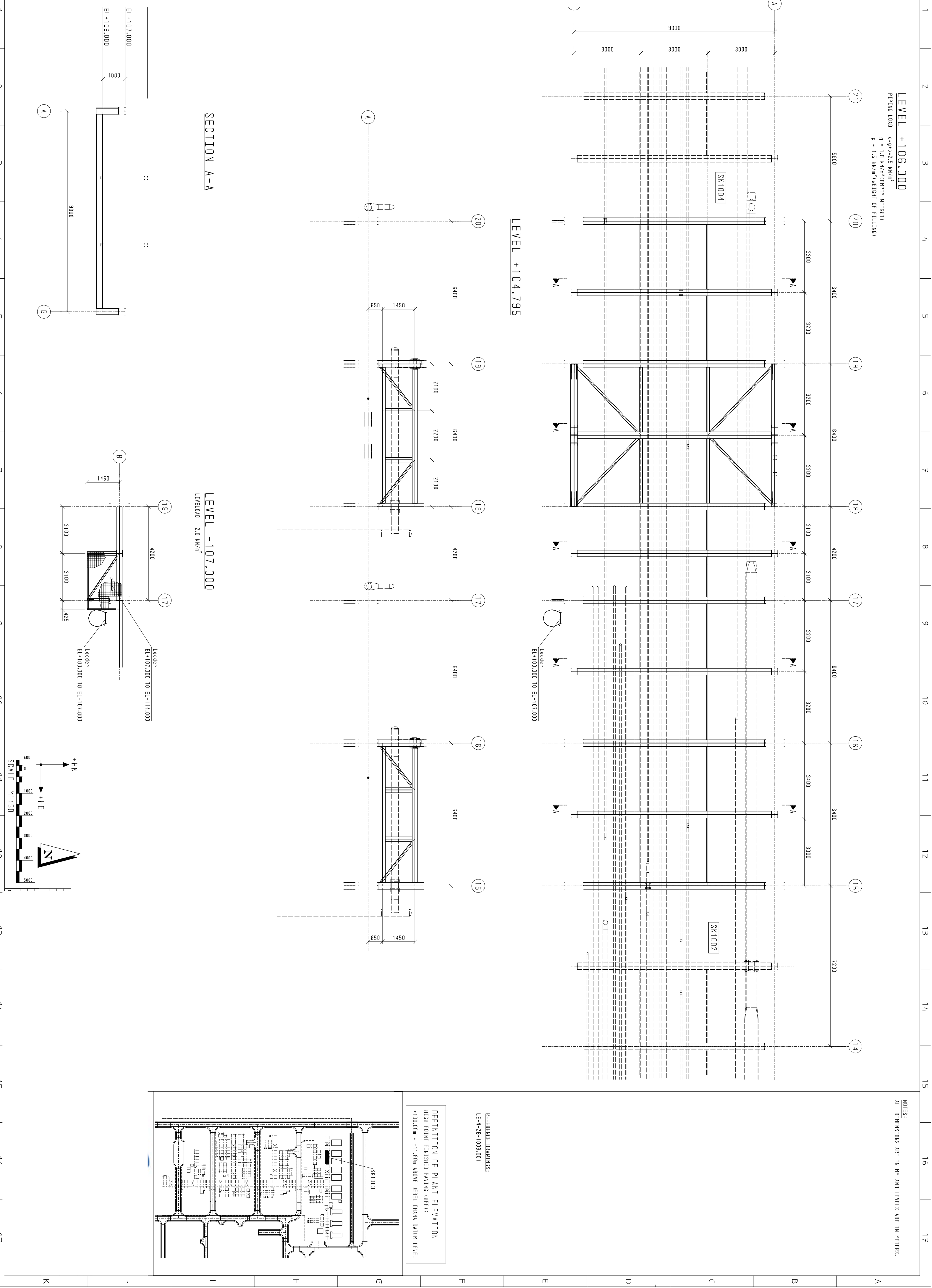
Problem formulation of Pipe rack Structure

A.1 Plan and Elevation of Pipe rack Structure

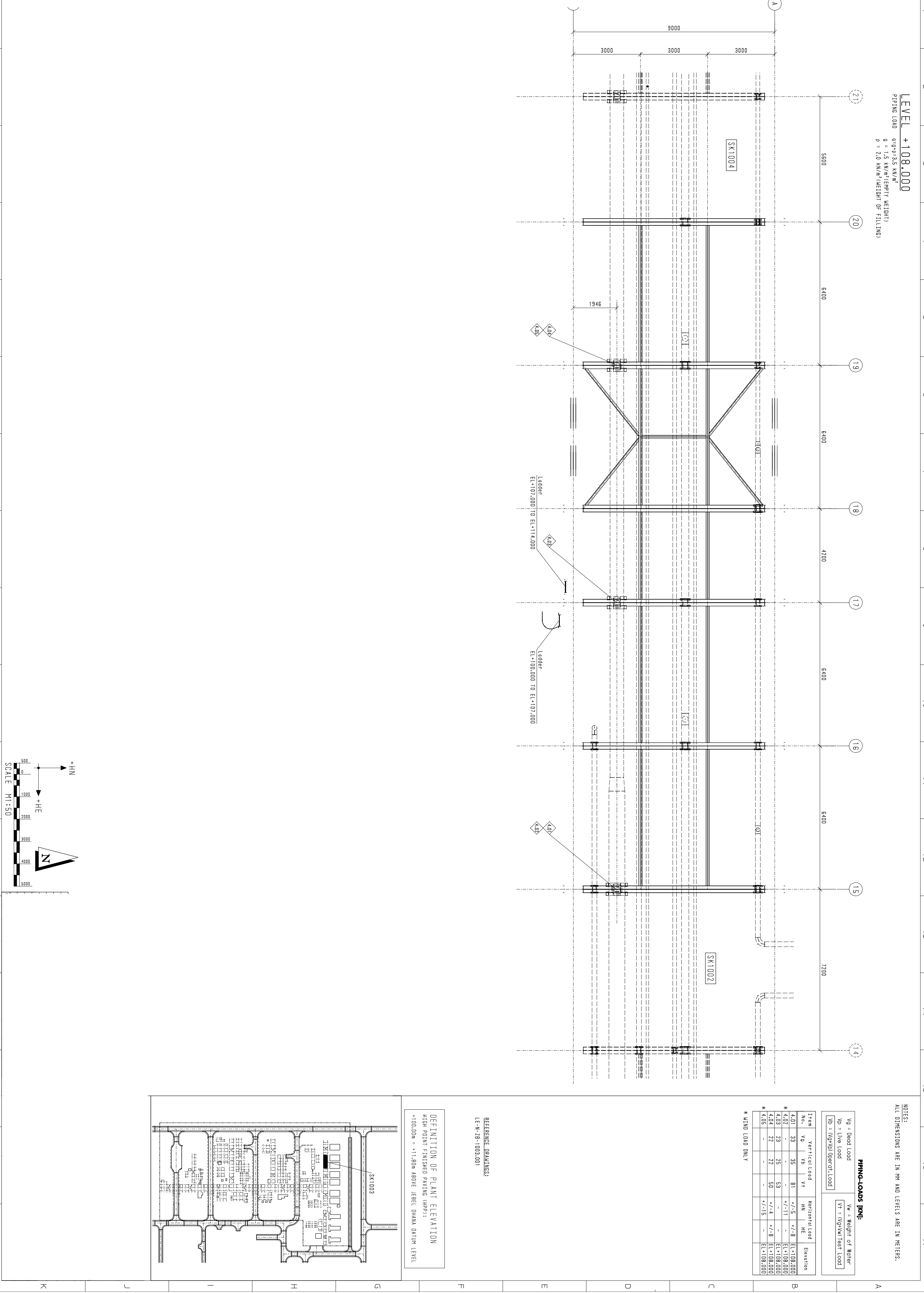
APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 124



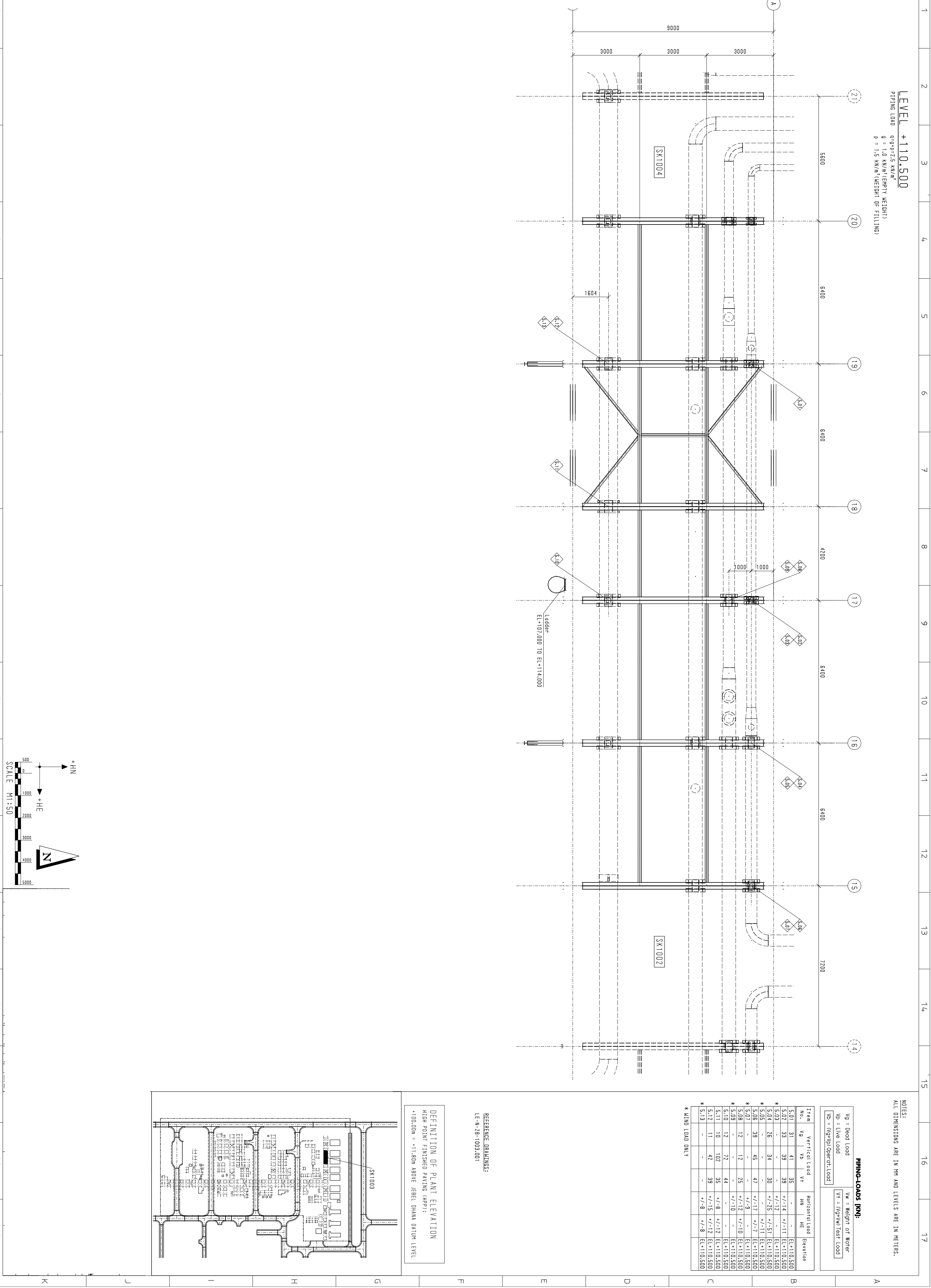
APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 125



APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 126



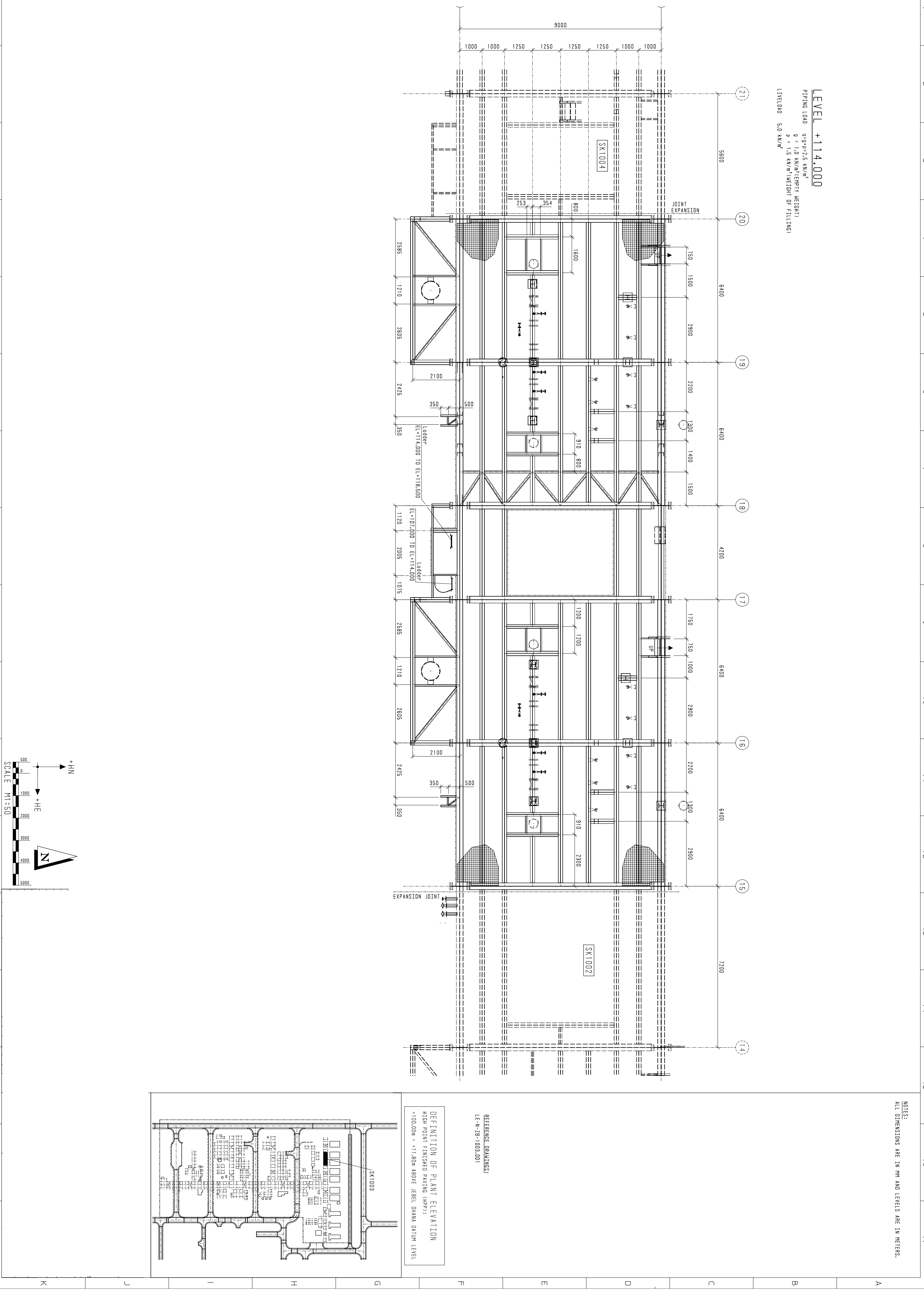
APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 127



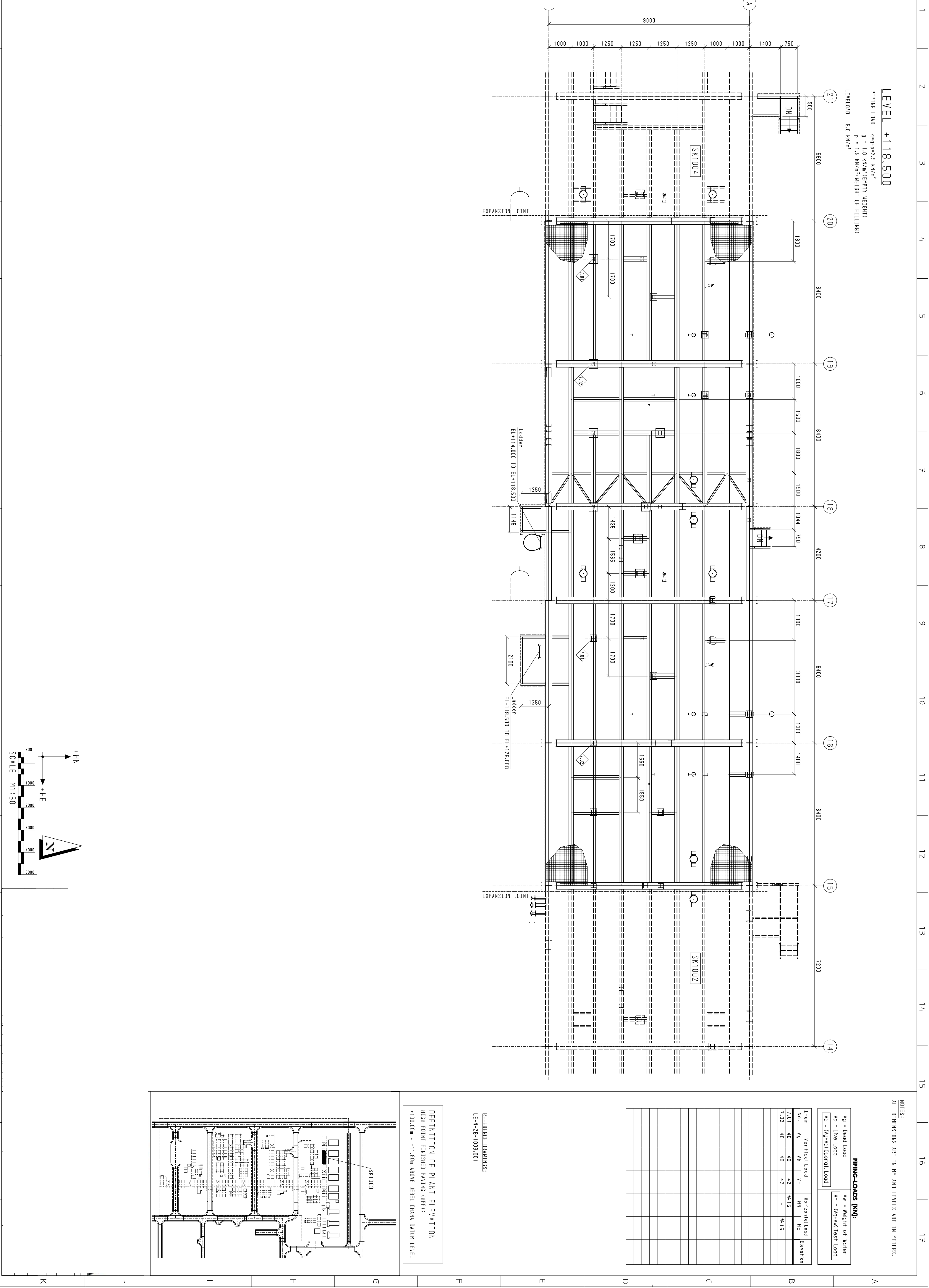
Item No.	Vertical Load Vg	Vertical Load Vb	Vertical Load Vt	Horizontal Load HN	Horizontal Load HE	Elevation
5.01	31	41	35	-	-	EL+110.500
5.02	33	39	39	+/-14	+/-11	EL+110.500
* 5.03	-	-	-	+/-12	-	EL+110.500
5.04	26	34	30	+/-25	+/-51	EL+110.500
* 5.05	-	-	-	-	+/-11	EL+110.500
5.06	39	45	47	+/-17	+/-7	EL+110.500
* 5.07	-	-	-	+/-9	-	EL+110.500
5.08	12	12	25	+/-12	+/-10	EL+110.500
* 5.09	-	-	-	+/-10	-	EL+110.500
5.10	12	72	44	-	-	EL+110.500
5.11	10	102	35	+/-8	+/-12	EL+110.500
5.12	11	42	39	+/-15	+/-12	EL+110.500
* 5.13	-	-	-	+/-8	+/-8	EL+110.500

* WIND LOAD ONLY

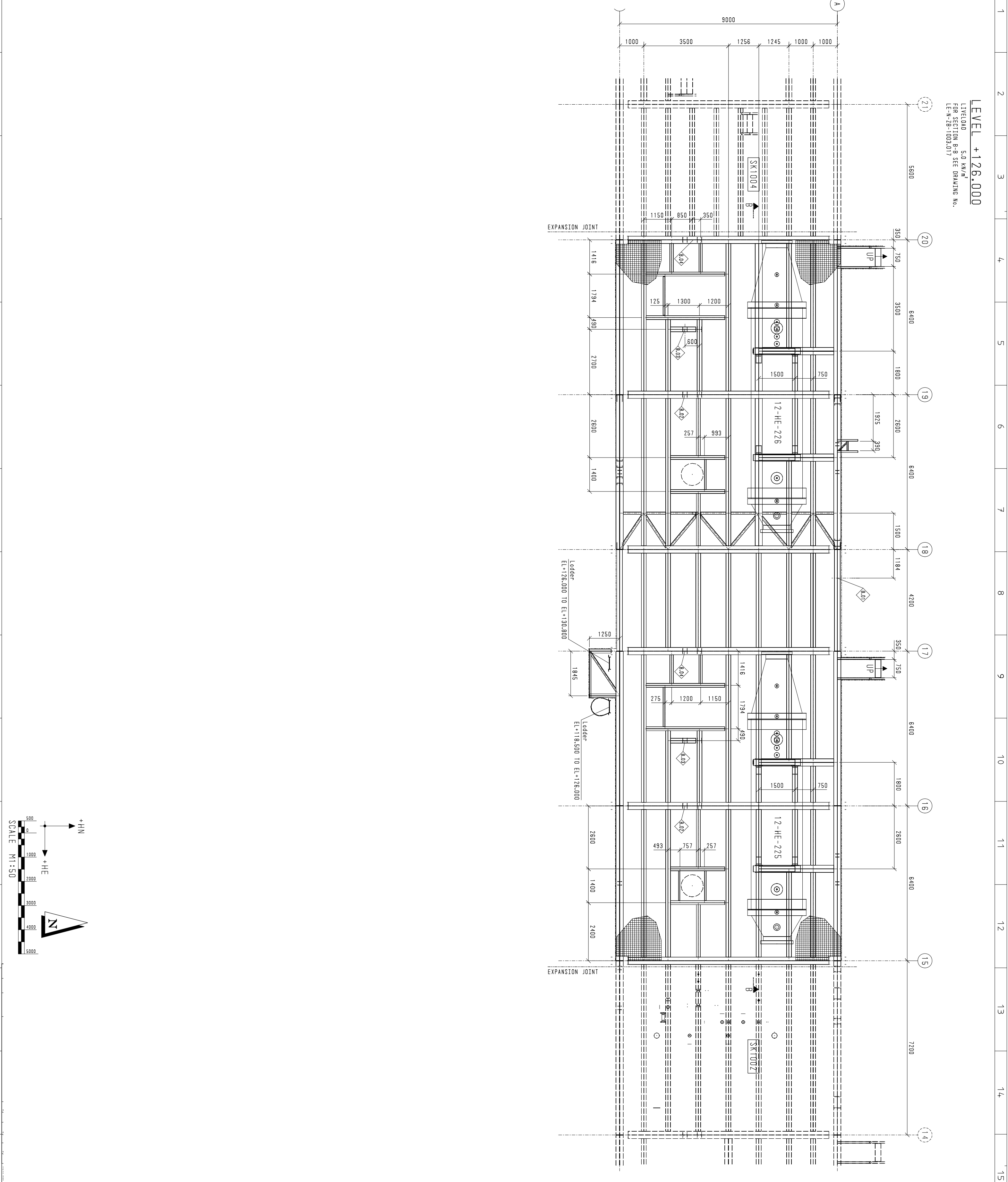
APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 128

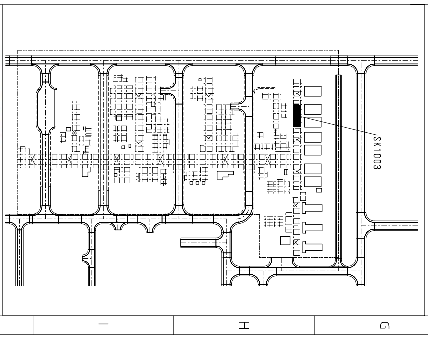


APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 129



APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 130





DEFINITION OF PLANT ELEVATION
HIGH POINT FINISHED PAVING (HPP):
+100.00m = +11.80m ABOVE JEBEL DHANA DATUM LEVEL

REFERENCE DRAWINGS:
LE-N-ZB-1003.001

NOTES:
ALL DIMENSIONS ARE IN MM AND LEVELS ARE IN METERS.

EQUIPMENT-LOADS [KN]:

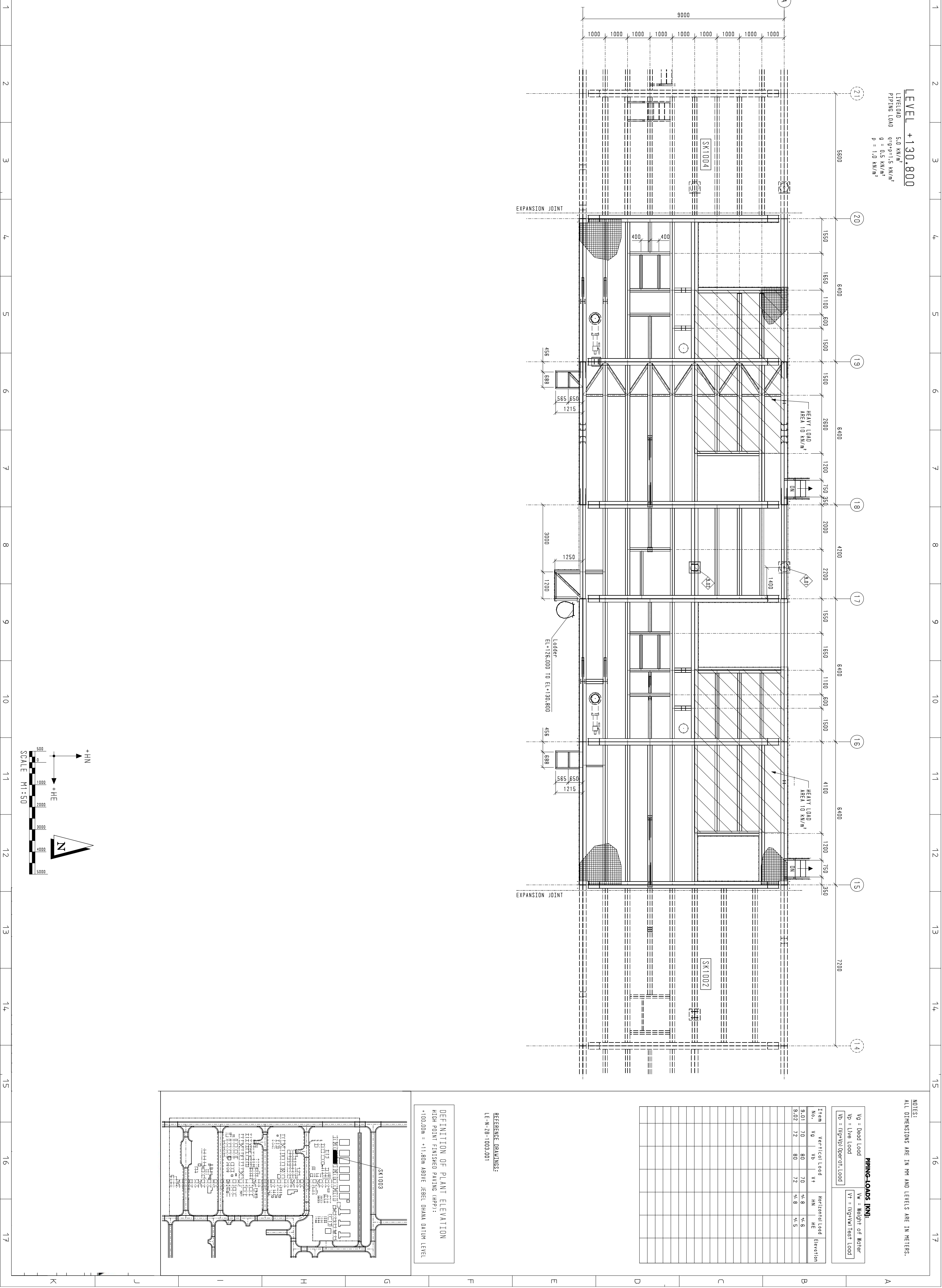
Equip. No.	Vertical Load Vg	Vertical Load Vb	Vertical Load Vt	Horizontal Load HN	Horizontal Load HE	Elevation
12-HE-225	1274	1500	1650	-	-	EL+126.000
12-HE-226	1274	1500	1650	-	-	EL+126.000

PIPING-LOADS [KN]:

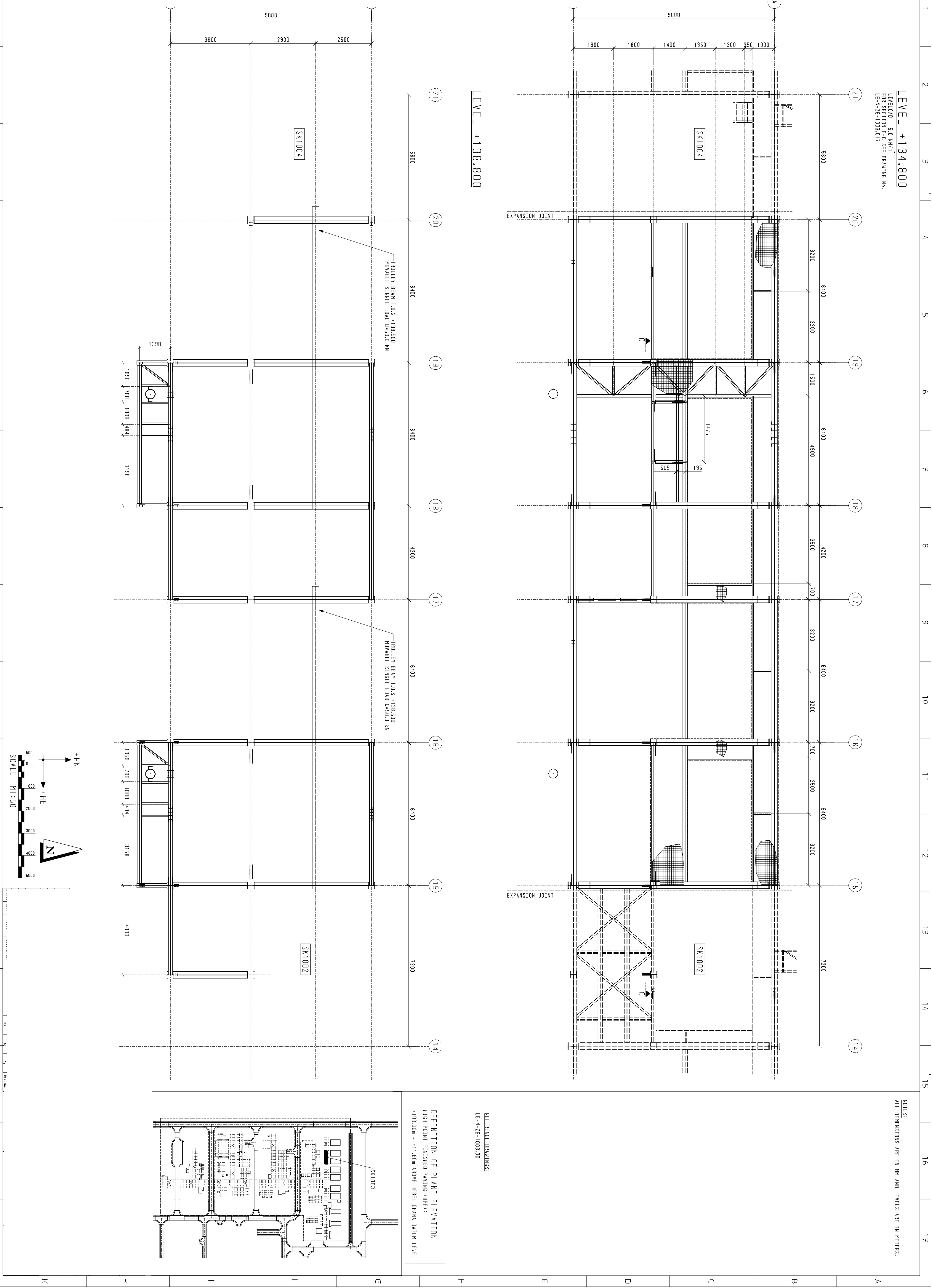
Vg = Dead Load
Vp = Live Load
Vb = (Vg+Vp) Operat. Load
Vw = Weight of Water
Vt = (Vg+Vw) Test Load

Item No.	Vertical Load Vg	Vertical Load Vb	Vertical Load Vt	Horizontal Load HN	Horizontal Load HE	Elevation
8.01	115	-	185	+/- 5	+/-18	
8.02	53	53	53	+/- 2	+/- 2	
8.03	130	130	135	+/-12	+/- 4	
8.04	67	67	78	+/- 4	+/- 4	

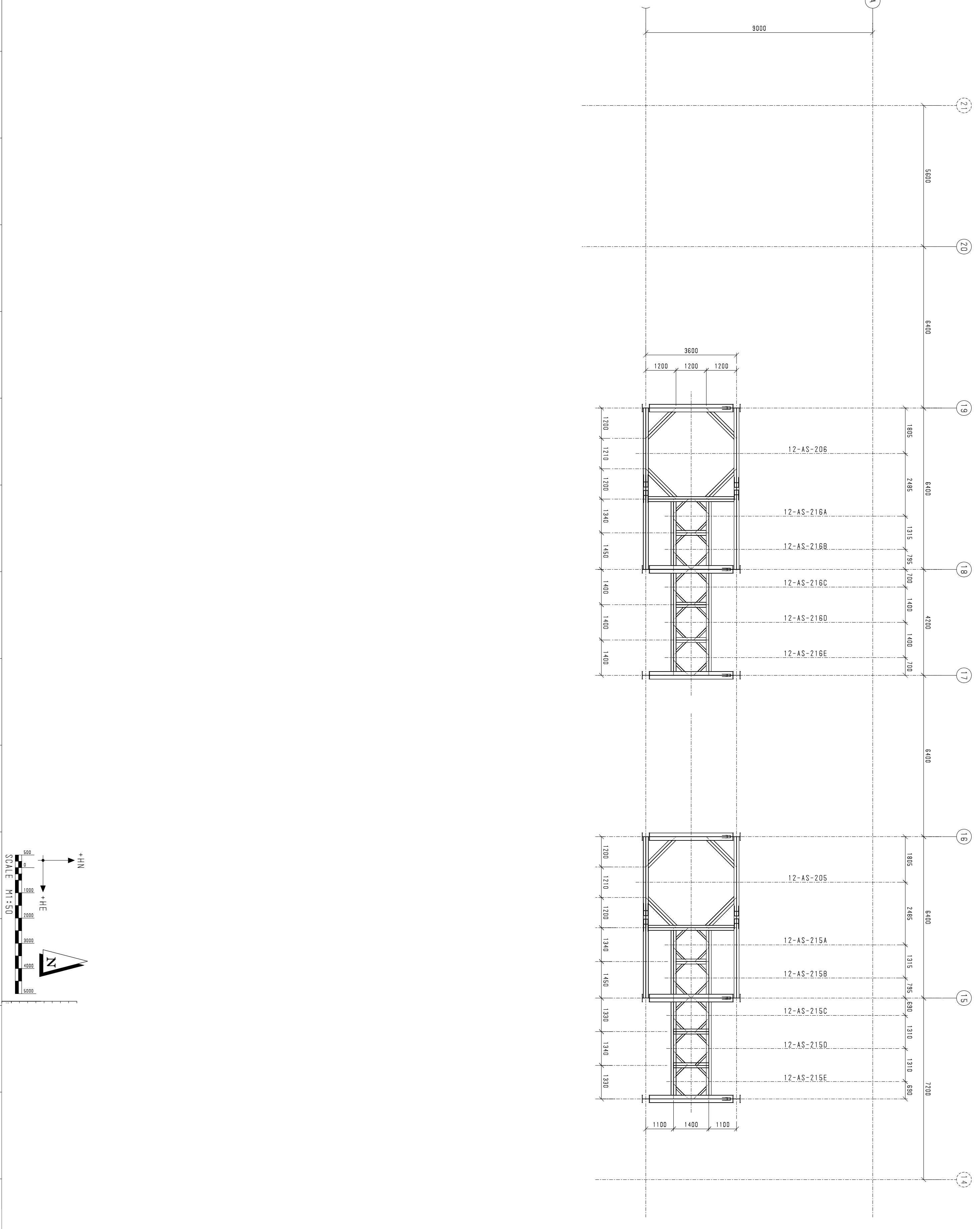
APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 131



APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 132



APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 133



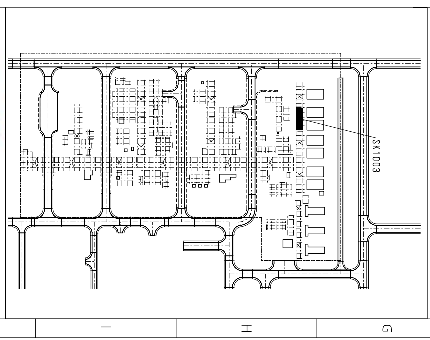
NOTES:
ALL DIMENSIONS ARE IN MM AND LEVELS ARE IN METERS.

EQUIPMENT-LOADS [KN]:

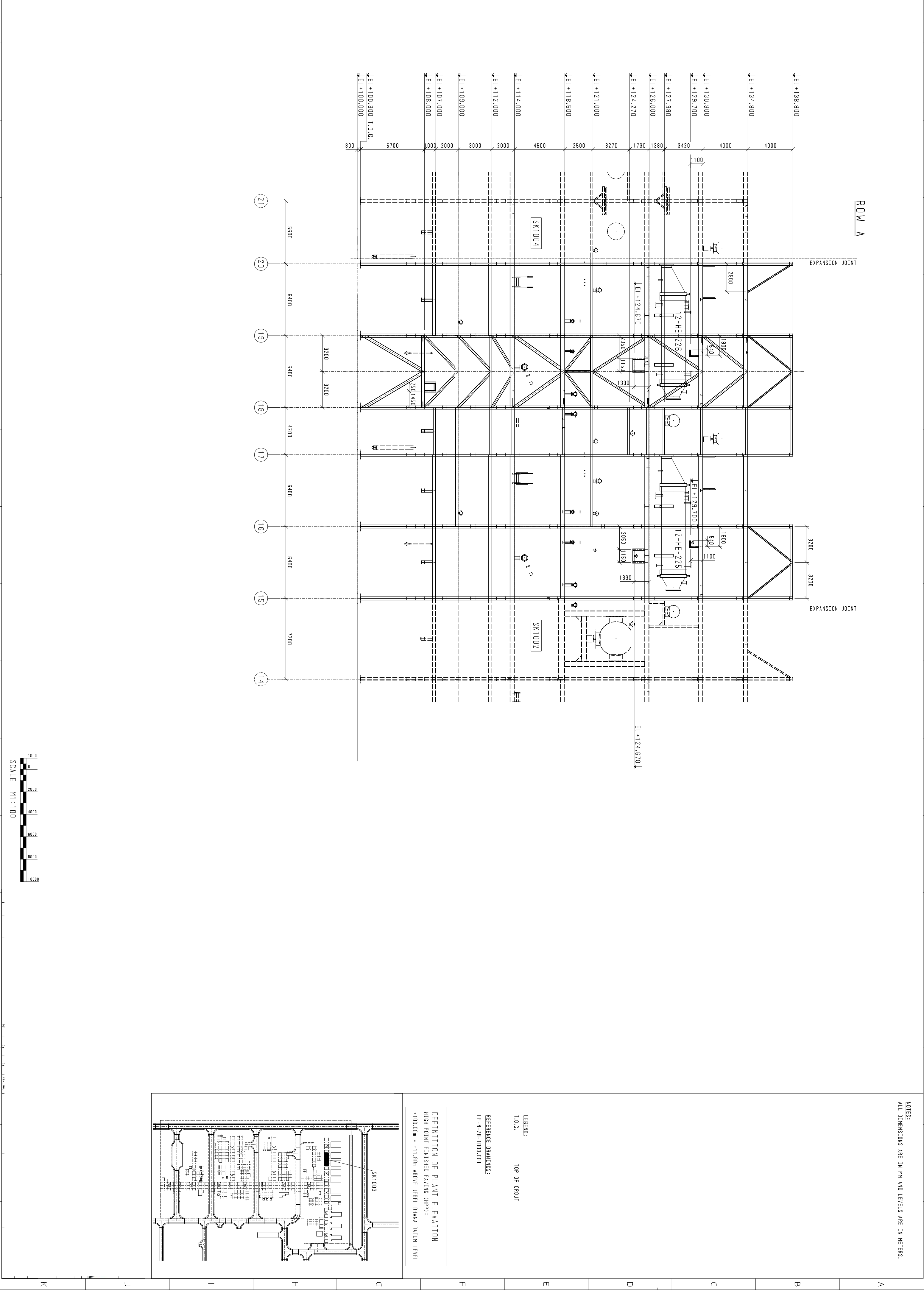
Equip. No.	Vertical Load			Horizontal Load		Elevation
	Vg	Vb	Vt	HN	HE	
12-AS-205	40	40	40	-	-	+142.300
12-AS-206	40	40	40	-	-	+142.300
12-AS-215A	5	5	5	-	-	+142.300
12-AS-215B	5	5	5	-	-	+142.300
12-AS-215C	5	5	5	-	-	+142.300
12-AS-215D	5	5	5	-	-	+142.300
12-AS-215E	5	5	5	-	-	+142.300
12-AS-216A	5	5	5	-	-	+142.300
12-AS-216B	5	5	5	-	-	+142.300
12-AS-216C	5	5	5	-	-	+142.300
12-AS-216D	5	5	5	-	-	+142.300
12-AS-216E	5	5	5	-	-	+142.300

REFERENCE DRAWINGS:
LE-N-ZB-1003.001

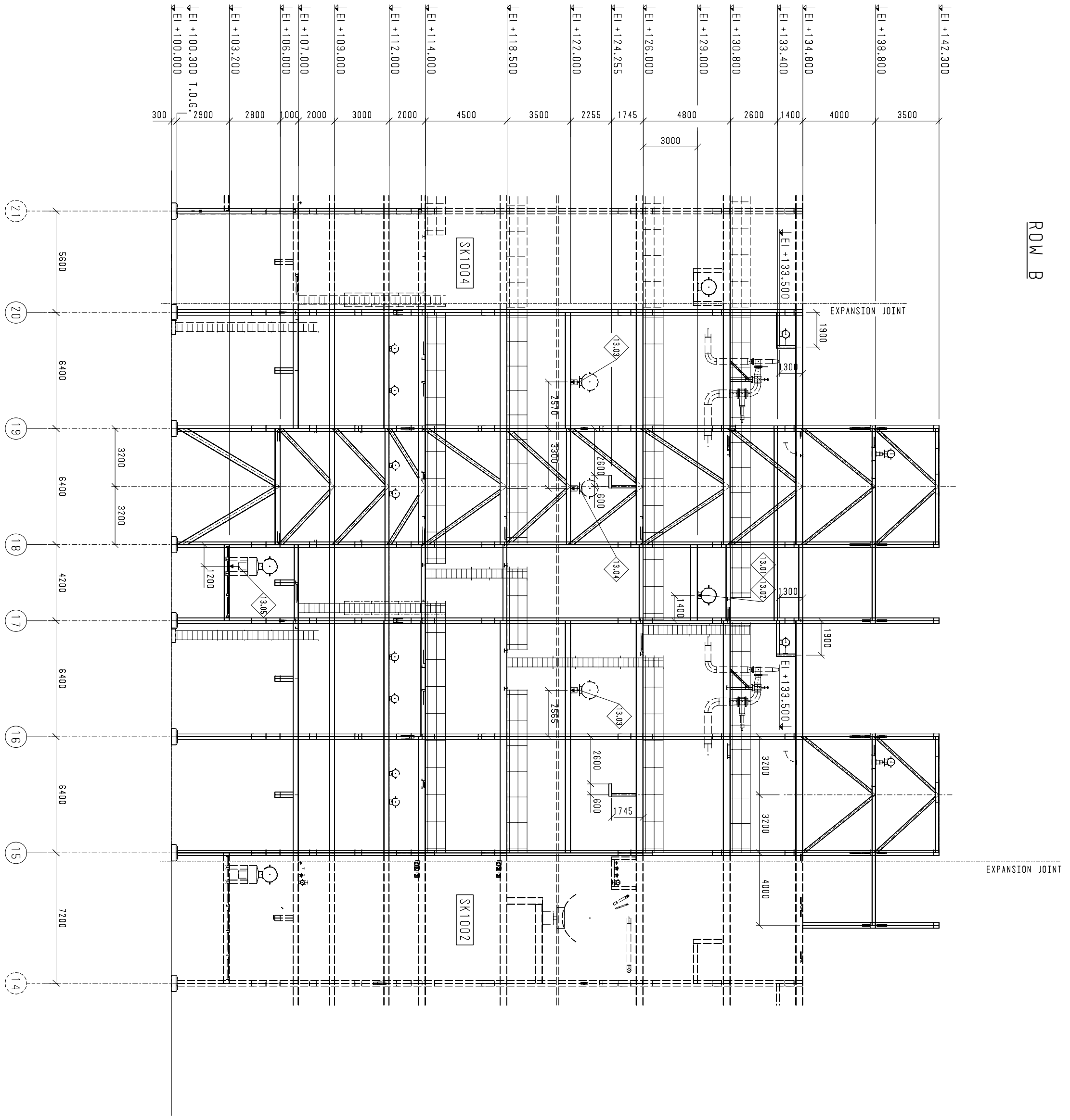
DEFINITION OF PLANT ELEVATION
HIGH POINT FINISHED PAVING (HPP):
+100.00m = +11.80m ABOVE JEBEL DHANA DATUM LEVEL

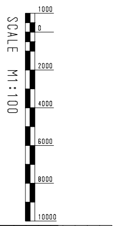


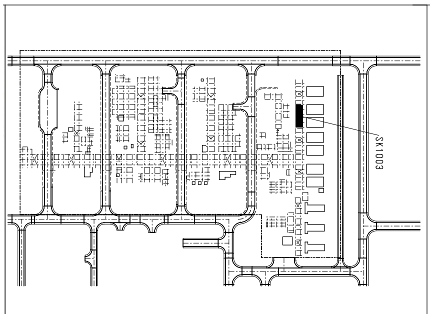
APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 134



APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 135







DEFINITION OF PLANT ELEVATION
HIGH POINT FINISHED PAVING (HPP):
+100.00m = +11.80m ABOVE JEBEL DHANA DATUM LEVEL

LEGEND:
T.O.G. TOP OF GROUT

REFERENCE DRAWING:
LE-N-ZB-1003.001

NOTES:
ALL DIMENSIONS ARE IN MM AND LEVELS ARE IN METERS.

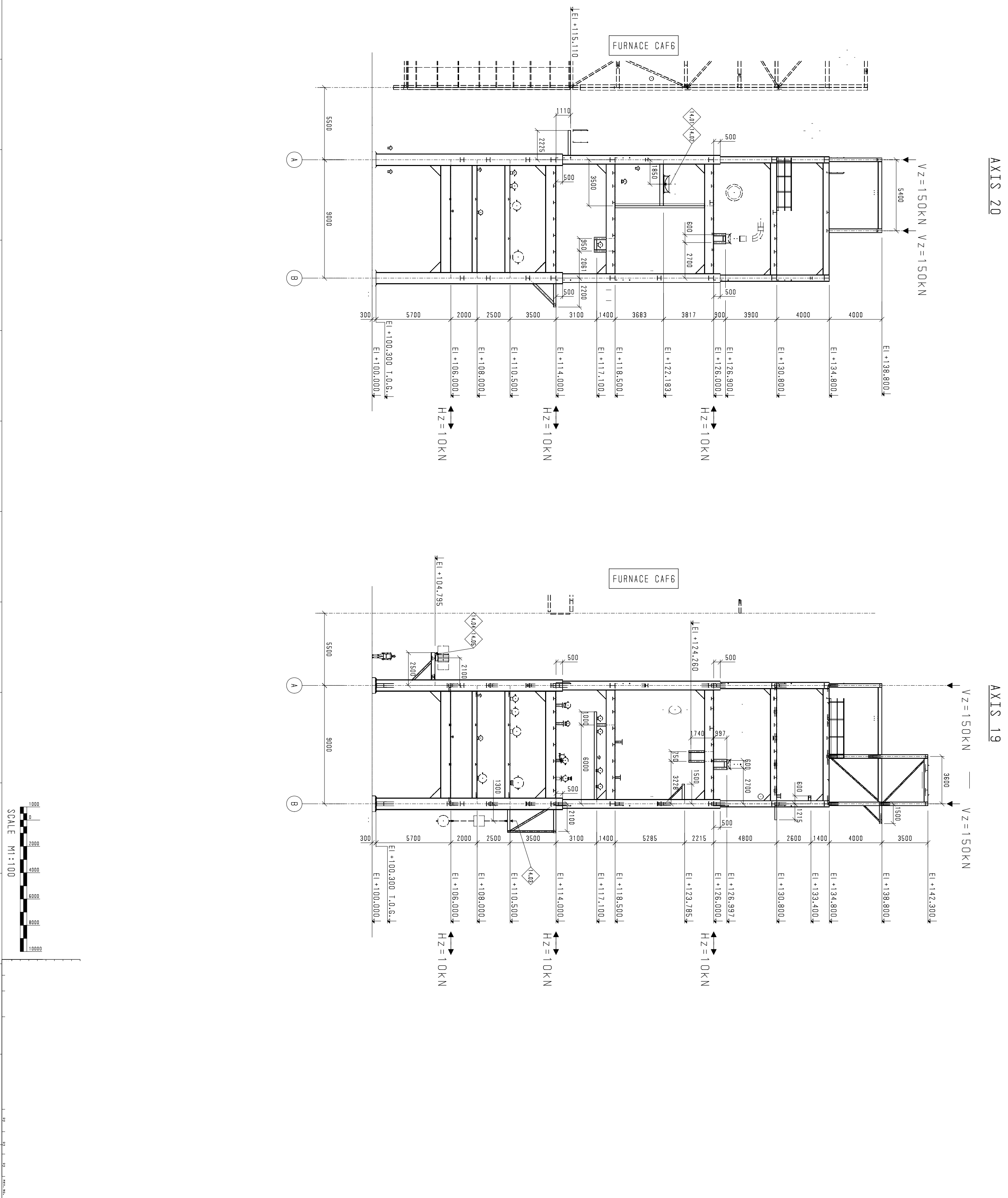
PIPING-LOADS [KN]:

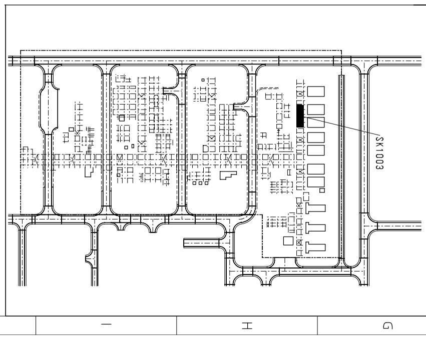
Vg = Dead Load
Vp = Live Load
Vb = (Vg+Vp) Operat. Load
Vw = Weight of Water
Vt = (Vg+Vw) Test Load

Item No.	Vertical Load Vg	Vertical Load Vb	Vertical Load Vt	Horizontal Load HN	Horizontal Load HE	Elevation
13.01	90	112	90	+/-17	+/-16	EL+129.000
* 13.02	-	-	-	-	+/-14	EL+129.000
13.03	32	35	32	+/-4	+/-4	EL+122.000
13.04	32	39	32	+/-6	+/-5	EL+122.000
13.05	30	30	30	+/-3	+/-3	EL+103.200

* WIND LOAD ONLY

APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 136





DEFINITION OF PLANT ELEVATION
HIGH POINT FINISHED PAVING (HPP):
+100.00m = +1.80m ABOVE JEBEL DHANA DATUM LEVEL

REFERENCE DRAWINGS:
LE-N-28-1003.001

LEGEND:
T.O.G. TOP OF GROUT

NOTES:
ALL DIMENSIONS ARE IN MM AND LEVELS ARE IN METERS.

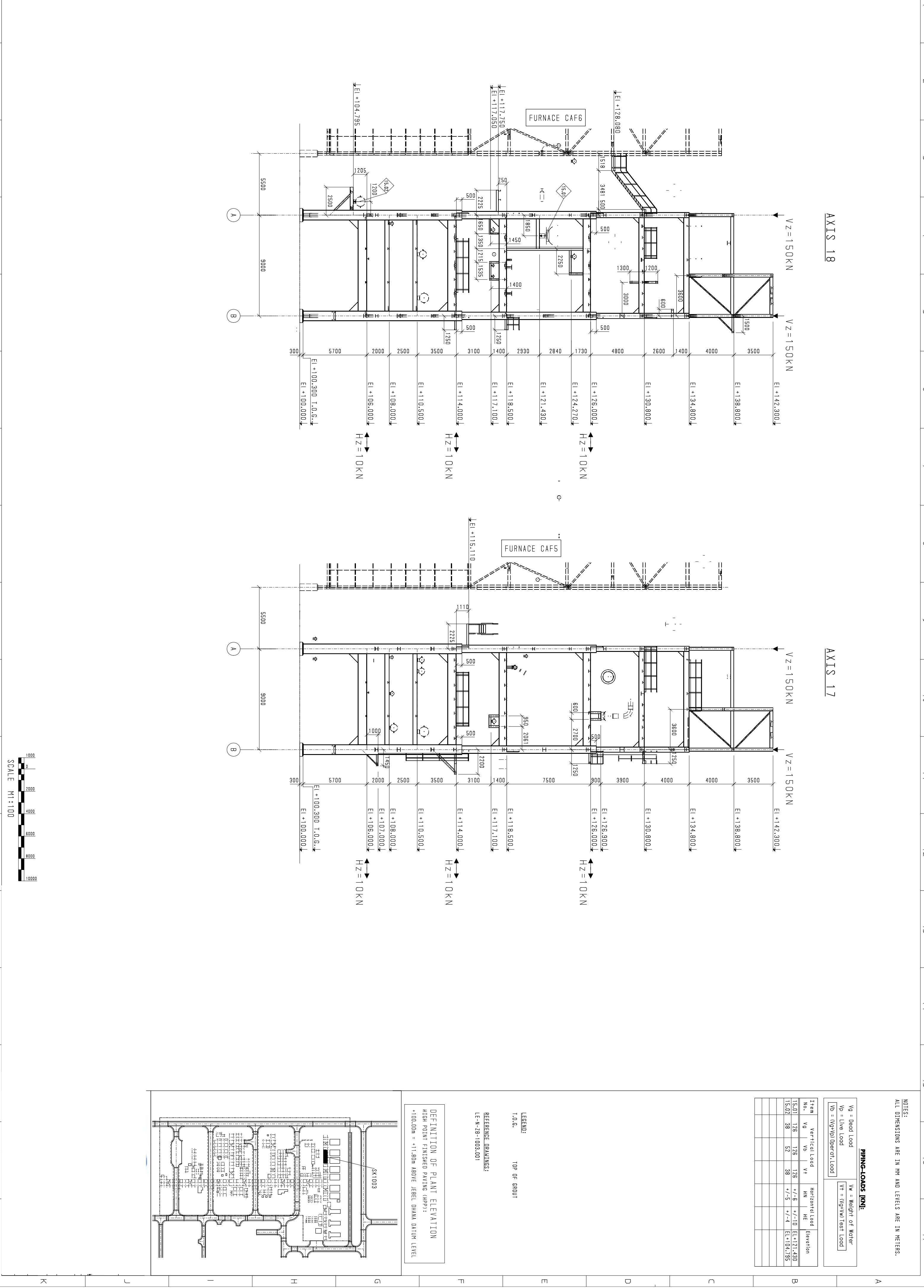
PIPING-LOADS [KN]:

Vg = Dead Load
Vp = Live Load
Vb = (Vg+Vp) Operat. Load
Vw = Weight of Water
Vt = (Vg+Vw) Test Load

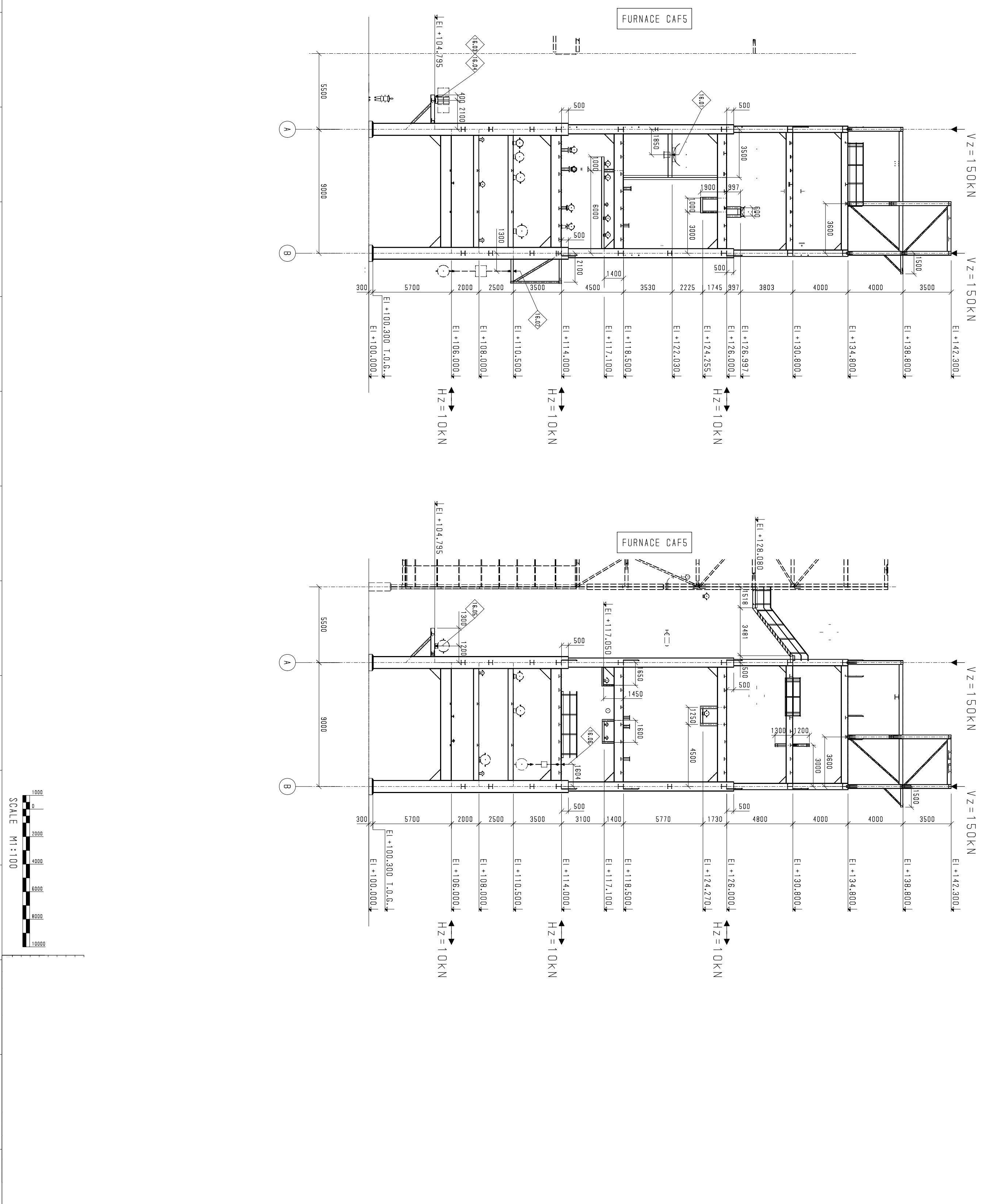
Item No.	Vertical Load Vg	Vertical Load Vb	Vertical Load Vt	Horizontal Load HN	Horizontal Load HE	Elevation
14.01	136	142	136	-	+/-14	EL+122.183
* 14.02	-	-	-	34	+/-10	EL+122.183
14.03	60	60	60	-	-	EL+110.500
14.04	45	85	45	+/-15	+/-11	EL+104.795
* 14.05	-	-	-	-	+/-15	EL+104.795

* WIND LOAD ONLY

APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 137



APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 138



NOTES:
ALL DIMENSIONS ARE IN MM AND LEVELS ARE IN METERS.

PIPING-LOADS [KN]:

Vg = Dead Load
Vp = Live Load
Vb = (Vg+Vp) Operat. Load
Vw = Weight of Water
Vt = (Vg+Vw) Test Load

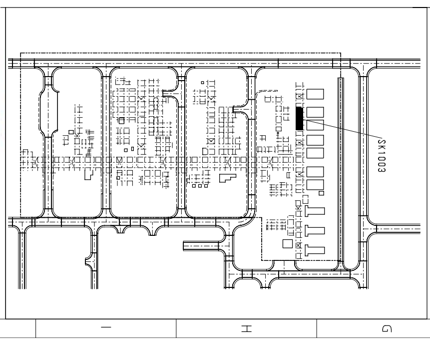
Item No.	Vertical Load Vg	Vb	Vt	Horizontal Load HN	HE	Elevation
16.01	89	89	89	+/-7	+/-4	EL+122.030
16.02	60	60	60	-	-	EL+110.500
16.03	45	85	45	+/-15	+/-11	EL+104.795
* 16.04	-	-	-	-	+/-15	EL+104.795
16.05	38	52	38	+/-5	+/-4	EL+104.795
16.06	11	68	40	-	-	EL+114.000

* WIND LOAD ONLY

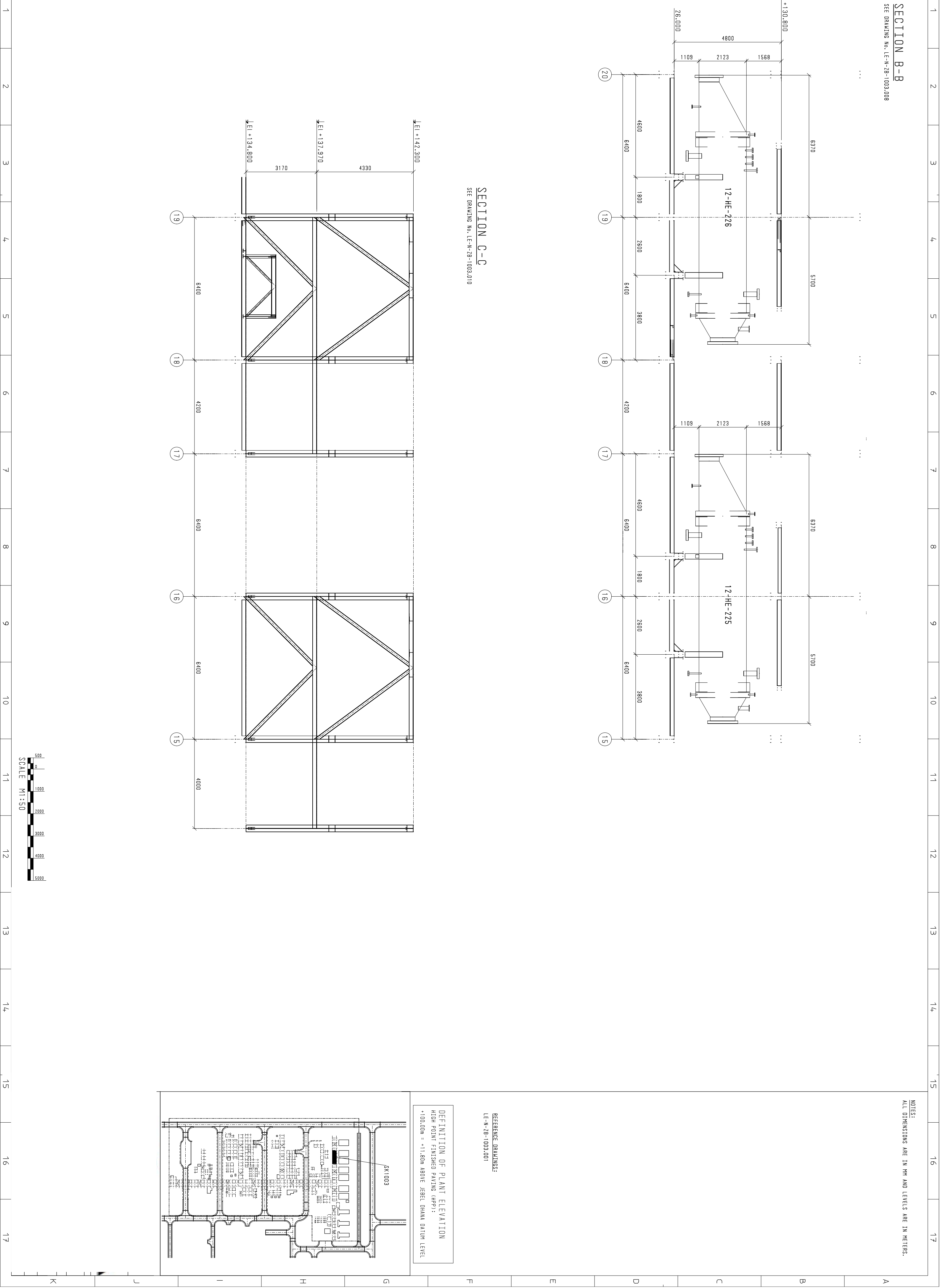
LEGEND:
T.O.G. TOP OF GROUT

REFERENCE DRAWINGS:
LE-N-ZB-1003.001

DEFINITION OF PLANT ELEVATION
HIGH POINT FINISHED PAVING (HPP):
+100.00m = +11.80m ABOVE JEBEL DHANA DATUM LEVEL



APPENDIX A. PROBLEM FORMULATION OF PIPE RACK STRUCTURE 139



A.2 Piping Load

1. Load Calculation for Empty Weight of Piping

1.) At EL +106.00

Piping Empty Area Load = **1** kN/m²

On Axis	Span	UDL (kN/m)
15	3.4	**3.4**
Betn 15 & 16	3.2	**3.2**
16	3.2	**3.2**
Betn 16 & 17	3.2	**3.2**
17	2.65	**2.65**
Betn 17 & 18	2.1	**2.1**
18	2.65	**2.65**
Betn 18 & 19	3.2	**3.2**
19	3.2	**3.2**
Betn 19 & 20	3.2	**3.2**
20	3	**3**

2.) At EL +108.00

Piping Empty Area Load = **1.5** kN/m²

On Axis	Span	UDL (kN/m)
15	6.8	**10.2**
16	6.4	**9.6**
17	5.3	**7.95**
18	5.3	**7.95**
19	6.4	**9.6**
20	6	**9**

3.) At EL +110.50

Piping Empty Area Load = **1** kN/m²

On Axis	Span	UDL (kN/m)
15	6.8	**6.8**
16	6.4	**6.4**
17	5.3	**5.3**
18	5.3	**5.3**
19	6.4	**6.4**
20	6	**6**

4.) At EL +114.00

Piping Empty Area Load = **1** kN/m²

On Axis	Span	UDL (kN/m)
15	6.8	**6.8**
16	6.4	**6.4**
17	5.3	**5.3**
18	5.3	**5.3**
19	6.4	**6.4**
20	6	**6**

5.) At EL +118.500

Piping Empty Area Load = **1** kN/m²

On Axis	Span	UDL (kN/m)
15	6.8	**6.8**
16	6.4	**6.4**
17	5.3	**5.3**
18	5.3	**5.3**
19	6.4	**6.4**
20	6	**6**

6.) At EL +130.80

Piping Empty Area Load = **0.5** kN/m²

On Axis	Span	UDL (kN/m)
15	6.8	**3.4**
16	6.4	**3.2**
17	5.3	**2.65**
18	5.3	**2.65**
19	6.4	**3.2**
20	6	**3**

2. Load Calculation for Operating Weight of Piping

1.) At EL +106.00

Piping Empty Area Load = 1.5 kN/m^2

On Axis	Span	UDL (kN/m)
15	3.4	5.1
Betn 15 & 16	3.2	4.8
16	3.2	4.8
Betn 16 & 17	3.2	4.8
17	2.65	3.975
Betn 17 & 18	2.1	3.15
18	2.65	3.975
Betn 18 & 19	3.2	4.8
19	3.2	4.8
Betn 19 & 20	3.2	4.8
20	3	4.5

2.) At EL +108.00

Piping Empty Area Load = 2 kN/m^2

On Axis	Span	UDL (kN/m)
15	6.8	13.6
16	6.4	12.8
17	5.3	10.6
18	5.3	10.6
19	6.4	12.8
20	6	12

3.) At EL +110.50

Piping Empty Area Load = 1.5 kN/m^2

On Axis	Span	UDL (kN/m)
15	6.8	10.2
16	6.4	9.6
17	5.3	7.95
18	5.3	7.95
19	6.4	9.6
20	6	9

4.) At EL +130.80

Piping Empty Area Load = 1.5 kN/m^2

On Axis	Span	UDL (kN/m)
15	6.8	10.2
16	6.4	9.6
17	5.3	7.95
18	5.3	7.95
19	6.4	9.6
20	6	9

3. Load Calculation for Test Weight of Piping

1.) At EL +104.00

Piping Empty Area Load = 1.5 kN/m^2

On Axis	Span	UDL (kN/m)
15	3.4	5.1
Betn 15 & 16	3.2	4.8
16	3.2	4.8
Betn 16 & 17	3.2	4.8
17	2.65	3.975
Betn 17 & 18	2.1	3.15
18	2.65	3.975
Betn 18 & 19	3.2	4.8
19	3.2	4.8
Betn 19 & 20	3.2	4.8
20	3	4.5

2.) At EL +106.00

Piping Empty Area Load = 1.5 kN/m^2

On Axis	Span	UDL (kN/m)
15	3.4	5.1
Betn 15 & 16	3.2	4.8
16	3.2	4.8
Betn 16 & 17	3.2	4.8
17	2.65	3.975
Betn 17 & 18	2.1	3.15
18	2.65	3.975
Betn 18 & 19	3.2	4.8
19	3.2	4.8
Betn 19 & 20	3.2	4.8
20	3	4.5

3.) At EL +108.00

Piping Empty Area Load = 2 kN/m^2

On Axis	Span	UDL (kN/m)
15	6.8	13.6
16	6.4	12.8
17	5.3	10.6
18	5.3	10.6
19	6.4	12.8
20	6	12

4.) At EL +110.50

Piping Empty Area Load = 1.5 kN/m^2

On Axis	Span	UDL (kN/m)
15	6.8	10.2
16	6.4	9.6
17	5.3	7.95
18	5.3	7.95
19	6.4	9.6
20	6	9

A.2.1 Wind Load Calculation

1. Force Cofficient (C_f) Calculation As Per ASCE-07-2005[22]

For Wind Force In N-S Direction

Width of Frame (m) :	29.80	Gross Area of Frame (m^2) :	1037.04
Height of Frame (m) :	34.80	Solid Area of Frame (m^2) :	152.78
		Solidity Ratio (ε) :	**0.15**

Calculation of Solid Area of Frame

Member				Af
Mark	L (m)	B (m)	No.	m²
Columns upto level 106.5	6.50	0.31	6	12.090
Columns upto level 114.50	8.00	0.30	6	14.400
Columns upto level 126.50	12.00	0.30	6	21.600
Columns upto level 134.8	8.30	0.30	6	14.940
Beam at EL+107.00 Level	29.80	0.30	1	8.940
Beam at EL+109.00 Level	29.80	0.24	1	7.152
Beam at EL+112.00 Level	29.80	0.24	1	7.152
Beam at EL+114.00Level	29.80	0.24	1	7.152
Beam at EL+118.50 Level	29.80	0.24	1	7.152
Beam at EL+126.00 Level	29.80	0.70	1	20.860
Beam at EL+130.80 Level	29.80	0.20	1	5.960
Beam at EL+134.80 Level	29.80	0.20	1	5.960
Due to Bracing	6.50	0.26	2	3.380
	4.40	0.20	2	1.760
	4.40	0.20	2	1.760
	3.80	0.20	2	1.520
	5.50	0.20	2	2.200
	4.06	0.20	2	1.624
	2.50	0.20	1	0.500
	5.90	0.20	2	2.360
	5.70	0.20	2	2.280
	5.10	0.20	2	2.040
		Solid Area of Frame		152.782

2. Force Cofficient (C_f) Calculation As Petrochemical Guidelines ASCE

For Wind Force In E-W Direction

Force Coeff. (C_f) :	1.80	Reference Fig. 6-22 of ASCE 7-05	
Width of Frame (m) :	10.00	Gross Area of Frame (m^2) :	339.04
Height of Frame (m) :	34.80	Solid Area of Frame (m^2) :	93.00
		Solidity Ratio (ε) :	**0.27**

Calculation of Solid Area of Frame

Member				Af
Mark	L (m)	B (m)	Nos	m²
Columns upto 106.50	6.50	1.00	2	13.000
Columns upto 114.50	8.00	0.90	2	14.400
Columns upto 126.50	12.00	0.70	2	16.800
Columns upto 134.8	8.30	0.50	2	8.300
Beam at EL+106.00 Level	9.00	0.90	1	8.100
Beam at EL+108.00 Level	9.00	0.30	1	2.700
Beam at EL+110.50 Level	9.00	0.30	1	2.700
Beam at EL+114.00 Level	9.00	0.70	1	6.300
Beam at EL+118.50 Level	9.00	0.70	1	6.300
Beam at EL+126.00 Level	9.00	0.70	1	6.300
Beam at EL+130.8 Level	9.00	0.45	1	4.050
Beam at EL+134.80 Level	9.00	0.45	1	4.050
		Solid Area of Frame		93.000

does not provide any method for calculation for C_f incorporating shielding effect; hence for calculation of C_f a report on "wind load on petrochemicals facilities" by

ASCE has been used for calculation.C_f is force co-efficient for the set of frames.

$C_f = C_{Dg} / \varepsilon$

Calculation of Wind Force as per Petrochemical Guidelines

Solidity Ratio (e)	=	0.27	
No.of Frames (N)	=	6	
S_F	=	6.4 m	(Where Sf is c/c frame spacing)
B	=	10 m	(Where, B is frame width measured outside to outside edge)
S_F/B	=	0.64	
a_{max}	=	31 degree	

For, N=3 , e = 0.1 & S_F/B =		0.64			
C_f	=	3.7			
			C_f,N=3	=	**3.25**
For, N=3 , e = 0.5 & S_F/B =		0.64			
C_f	=	2.65			

For, N=10 , e = 0.1 & S_F/B =		0.64			
C_f	=	10.6			
			C_f,N=10	=	**8.90**
For, N=10 , e = 0.5 & S_F/B =		0.64			
C_f	=	6.6			

Interpolating between
the two results for N=3 & N=10 for the case N=9
C_f = **5.7**

3. Wind Load in due to self obstruction (East West Direction)

Sr No.	Description			Height From Ground (m)	q_z (kN/m^2)
	Row A & B				
1	Level	106.00		6.00	0.95
				6.00	0.95
2	Level	108.00		8.00	1.01
				8.00	1.01
3	Level	110.50		10.50	1.05
				10.50	1.05
4	Level	114.00		14.00	1.12
				14.00	1.12
5	Level	118.50		18.50	1.18
				18.50	1.18
6	Level	126.00		26.00	1.28
				26.00	1.28
7	Level	130.80		30.80	1.30
				30.80	1.30
8	Level	134.80		34.80	1.34
				34.80	1.34
Cf	=	**1.80**			
9	Level	138.80		38.80	1.37
				38.80	1.37
10	Level	142.30		42.30	1.41
				42.30	1.41

Member	Width (W)	Depth (D)	Af	Wind Force $F = q_z \times G \times C_f \times A_f$ (kN)	Total wind force at each node
	(m)	(m)	(m^2)		(kN)
Beam	4.500	0.900	4.050	18.710	
column	6.000	1.000	6.000	27.720	46.430
Beam	4.500	0.300	1.350	6.600	
column	2.000	0.900	1.800	8.800	15.400
Beam	4.500	0.300	1.350	6.860	
column	2.500	0.900	2.250	11.430	18.290
Beam	4.500	0.700	3.150	17.120	
column	3.500	0.900	3.150	17.120	34.240
Beam	4.500	0.700	3.150	17.990	
column	4.500	0.700	3.150	17.990	35.980
Beam	4.500	0.700	3.150	19.470	
column	7.500	0.700	5.250	32.440	51.910
Beam	4.500	0.450	2.025	12.800	
column	4.800	0.450	2.160	13.650	26.450
Beam	4.500	0.450	2.025	13.130	
column	4.000	0.450	1.800	11.680	24.810
Beam	4.500	0.200	0.900	1.890	
column	4.000	0.200	0.800	1.680	3.570
Beam	4.500	0.200	0.900	1.950	
column	3.500	0.200	0.700	1.520	3.470

4. Wind Force Calculation due to self obstruction (N-S Direction)

						Wind on axis 15			
Sr No.	Beam Elevations	Height From Ground (m)	qz (kN/m2)	Member			Af (m^2)	Wind Force $F = q_z \times G \times C_f \times A_f$ (kN)	Total wind force at node
	EL				B (m)	L (m)			kN
1	107.00	7.00	0.98	Beam	0.300	6.800	2.040	3.070	
		7.00	0.98	column	0.310	7.000	2.170	3.260	6.330
2	109.00	9.00	1.03	Beam	0.300	6.800	2.040	3.220	
		9.00	1.03	column	0.300	2.000	0.600	0.950	4.170
3	112.00	12.00	1.09	Beam	0.300	6.800	2.040	3.400	
		12.00	1.09	column	0.300	3.000	0.900	1.500	4.900
4	114.00	14.00	1.12	Beam	0.400	6.800	2.720	4.670	
		14.00	1.12	column	0.300	2.000	0.600	1.030	5.700
5	118.50	18.50	1.18	Beam	0.400	6.800	2.720	4.910	
		18.50	1.18	column	0.300	4.500	1.350	2.440	7.350
6	126.00	26.00	1.28	Beam	0.400	6.800	2.720	5.310	
		26.00	1.28	column	0.300	7.500	2.250	4.400	9.710
7	130.80	30.80	1.30	Beam	0.400	6.800	2.720	5.430	
		30.80	1.30	column	0.300	4.800	1.440	2.880	8.310
8	134.80	34.80	1.34	Beam	0.400	6.800	2.720	5.570	
		34.80	1.34	column	0.300	4.000	1.200	2.460	8.030
9	138.80	38.80	1.37	Beam	0.200	5.200	1.040	2.190	
		38.80	1.37	column	0.200	4.000	0.800	1.680	3.870
10	142.30	42.30	1.41	Beam	0.200	3.200	0.640	1.390	
		42.30	1.41	column	0.200	3.500	0.700	1.520	2.910

Wind on axis 16 , 19 , 20				Wind on axis 17 , 18			
	Af (m^2)	Wind Force $F = q_z \times G \times C_f \times A_f$ (kN)	Total wind force at node		Af (m^2)	Wind Force $F = q_z \times G \times C_f \times A_f$ (kN)	Total wind force at node
L (m)			kN	L (m)			kN
6.400	1.920	2.890		5.300	1.590	2.390	
7.000	2.170	3.260	6.150	7.000	2.170	3.260	5.650
6.400	1.920	3.030		5.300	1.590	2.510	
2.000	0.600	0.950	3.980	2.000	0.600	0.950	3.460
6.400	1.920	3.200		5.300	1.590	2.650	
3.000	0.900	1.500	4.700	3.000	0.900	1.500	4.150
6.400	2.560	4.400		5.300	2.120	3.640	
2.000	0.600	1.030	5.430	2.000	0.600	1.030	4.670
6.400	2.560	4.620		5.300	2.120	3.830	
4.500	1.350	2.440	7.060	4.500	1.350	2.440	6.270
6.400	2.560	5.000		5.300	2.120	4.140	
7.500	2.250	4.400	9.400	7.500	2.250	4.400	8.540
6.400	2.560	5.110		5.300	2.120	4.240	
4.800	1.440	2.880	7.990	4.800	1.440	2.880	7.120
6.400	2.560	5.250		5.300	2.120	4.350	
4.000	1.200	2.460	7.710	4.000	1.200	2.460	6.810
3.200	0.640	1.350		5.300	1.060	2.230	
4.000	0.800	1.680	3.030	4.000	0.800	1.680	3.910
3.200	0.640	1.390		5.300	1.060	2.300	
3.500	0.700	1.520	2.910	3.500	0.700	1.520	3.820

5. Wind Load on piping (N-S Direction)

Wind load on piping in North Southt direction

Cf	=	0.8	Fig.6-22 of ASCE 7-05	
G	=	1		
Contributing depth of pipe considered as		2		m

					Wind on axis 15			Wind on axis 16 , 19 , 20			Wind on axis 17 , 18		
Sr No.	Beam Elevations	Height From Ground (m)	q_z (kN/m^2)	Member		Af (m^2)	Wind Force $F = q_z \times G \times C_f \times A_f$ (kN)		Af (m^2)	Wind Force $F = q_z \times G \times C_f \times A_f$ (kN)		Af (m^2)	Wind Force $F = q_z \times G \times C_f \times A_f$ (kN)
	EL			Mark	L (m)			L (m)			L (m)		
1	114.00	14.00	1.12	Pipe	6.800	13.600	12.21	6.400	12.800	11.49	5.300	10.600	9.52
2	118.50	18.50	1.18	Pipe	6.800	13.600	12.83	6.400	12.800	12.07	5.300	10.600	10.00
3	126.00	26.00	1.28	Pipe	6.800	13.600	13.88	6.400	12.800	13.06	5.300	10.600	10.82
4	130.80	30.80	1.30	Pipe	6.800	13.600	14.19	6.400	12.800	13.36	5.300	10.600	11.06
5	134.80	34.80	1.34	Pipe	6.800	13.600	14.57	6.400	12.800	13.71	5.300	10.600	11.35

6. Wind Load on Bracing (N-S Direction)

Wind on Bracing in North South direction

Sr No.	Beam Elevations EL	Height From Ground (m)	q_z (kN/m^2)	Member	B (m)	L (m)	Af (m^2)	Wind Force $F = q_z \times G \times C_f \times A_f$ (kN)
					Wind on axis 12 & 13			
1	106.00	6.00	0.95	Bracing	0.300	6.550	1.965	2.870
2	109.00	9.00	1.03	Bracing	0.200	4.386	0.877	1.390
3	112.00	12.00	1.09	Bracing	0.200	4.386	0.877	1.460
4	114.00	14.00	1.12	Bracing	0.200	3.774	0.755	1.300
5	118.50	18.50	1.18	Bracing	0.200	5.522	1.104	2.000
6	126.00	26.00	1.28	Bracing	0.200	8.154	1.631	3.190
7	130.80	30.80	1.30	Bracing	0.200	5.769	1.154	2.310
8	134.80	34.80	1.34	Bracing	0.200	5.122	1.024	2.100
9	138.80	38.80	1.37	Bracing	0.200	5.122	1.024	2.150
10	142.30	42.30	1.41	Bracing	0.200	4.742	0.948	2.060

7. Wind load on equipments

Wind Force Calculation at level 142 m

Diameter of equipment	3 m	
Length of equipment	3.6 m	
Hieght of equipment	42 m	(From top of pedestal)
Equip supporting Beam EL	142.3 m	
Gust factor G	0.85	
Cf	0.8	
Shape factor	1.18	
Number of support on which equipment is resting =	4	
Length between two sopports of eqipment =	3.4 m	

Corresponding q_z 1.42 kN/m^2

<u>In North South direction & In East West direction</u>

Exposed area Af = length x diameter

= 3.6 x 3

= 10.8 m^2

Wind force = F = Af x qh x G x Cf x shape factor

= 10.8 x 1.42 x 0.85 x 0.8x1.18

= 12.31 kN

Wind force on each support = $\frac{12.31}{4}$ = 3.1 kN Say , 3.5 kN

Wind Force Calculation at Level 126 m

Diameter	2.123	m
Length	12.8	m
Hieght	26	m
Equip Supporting Beam EL+	126	m
Gust Factor G	0.85	
Cf	0.8	
Shape Factor	1.2	
Number of support for equipment =	2	

q_z = 1.28 kN/m^2

In longitudinal direction of equipment (North south direction)

Exposed area Af = length x diameter
= 12.8 x 2.123
= 27.18 m^2

Wind force = F = Af x qh x G x Cf x shape factor
= 27.18 x 1.28 x 0.85 x 0.8x1.2
= 28.39 kN

Wind force on each support = $\frac{28.39}{2}$ = 15 kN

In transverse direction of equipment (East West direction)

Exposed area Af = 3.14 x diameter ^ 2 / 4
= 3.14 x 2.123 ^ 2 / 4
= 3.54 m^2

Wind force = F = Af x qh x G x Cf x shape factor
= 3.54 x 1.28 x 0.85 x 0.8x1.2
= 3.70 kN

Wind force on fixed support = $\frac{3.70}{1.00}$ = 3.7 kN (say 4 kN)

A.3 Effective Length Factor K

level	AISC:360-05	IS:800-2007	IS:800 1984
All Grids - 100.3 to 106.0 m	2.60	2.60	2.60
All Grids - 106.0 to 114.0	1.78	1.80	1.80
All Grids - 114.0 to 118.5 m	1.86	1.88	1.88
All Grids - 118.5 to 126.00 m	1.69	1.70	1.70
All Grids - 126.0 to 130.8	1.77	1.77	1.77
All Grids - 130.80 to 134.8 m	2.03	2.10	2.10

A.4 Primary Load Cases and Load Combination

Load Case No.	Abbreviation	Load Title
1001		Self wt. of structure
1002		Dead load of secondary beams
1003		Dead load of grating on floor
1004		Dead load of secondary items
1	**DL**	**Dead load [Total]**
2001		Live load on main floors
2002		Live load on secondary platforms
2	**LL**	**Live load [Total]**
3001		Empty weight of piping (UDL))
3002		Empty weight of piping (UDL)
3003		Empty weight of equipment
3	$\mathbf{DL}_{empty}$	**Empty weight of piping/equipment [Total]**
4001		Operating weight of piping (UDL)
4002		Operating weight of piping (concentrated)
4003		Hydrotest weight of equipment
4004		Cable tray loads
4	$\mathbf{DL}_{OP}$	**Operating weight of piping/equipment [Total]**
5001		Hytrotest weight of piping (UDL)
5002		Hydrotest weight of piping (concentrated)
5003		Hydrotest weight of equipment
5	$\mathbf{DL}_{Test}$	**Hydrotest weight of piping/equipment [Total]**
6001		Wind load on structure due to self obstruction in x-(e-w) direction
6002		Wind load on grating in x-(e-w) direction
6003		Wind load on piping in x-(e-w) direction
6004		Wind load on equipment in x-(e-w) direction
6	$\mathbf{WL}_X$	**Wind in x-(E-W) direction**
7001		Wind load on structure due to self obstruction in z-(n-s) direction
7002		Wind load on grating in z-(n-s) direction
7003		Wind load on piping in z-(n-s) direction
7004		Wind load on equipment in z-(n-s) direction
7	$\mathbf{WL}_Z$	**Wind load in z-(N-S) direction**
10	$\mathbf{TL}_X$	**Pipe thermal load in x-(e-w) direction**
11	$\mathbf{TL}_Z$	**Pipe thermal load in z-(n-s) direction**
50	**Rv**	**Reserve load- vertical direction**
51	$\mathbf{R\_H}_E$	**Reserve load- horizontal in x-(e-w) direction**
52	$\mathbf{R\_H}_N$	**Reserve load- horizontal in z-(n-s) direction**

LOAD COMBINATIONS AS PER ASCE 7-05 FOR PIPERACK STRUCTURE (CHAPTER 7)

Load	Sr.	Load	1	2	3	4	6	7	10	11	12	50	51	52
Conditions	No.	Comb	DL	LL	DL_{empty}	DL_{op}	WLX	WLZ	TL_X	TL_Z	$TL_{Min}X$	R_V	R_{HE}	R_{HN}
OP.	1	301	1.40	-	-	1.40	-	-	1.40	1.40	-	1.40	1.40	1.40
	2	302	1.40	-	-	1.40	-	-	-1.40	-1.40	-	1.40	-1.40	-1.40
	3	303	1.40	-	-	1.40	-	-	1.40	-1.40	-	1.40	1.40	-1.40
	4	304	1.40	-	-	1.40	-	-	-1.40	1.40	-	1.40	-1.40	1.40
OP. + LL	5	305	1.20	1.60	-	1.20	-	-	1.20	1.20	-	1.20	1.20	1.20
	6	306	1.20	1.60	-	1.20	-	-	-1.20	-1.20	-	1.20	-1.20	-1.20
	7	307	1.20	1.60	-	1.20	-	-	-1.20	1.20	-	1.20	-1.20	1.20
	8	308	1.20	1.60	-	1.20	-	-	1.20	-1.20	-	1.20	1.20	-1.20
Test	9	309	1.40	-	-	-	-	-	-	-	-	-	-	-
Test+ LL	10	310	1.20	1.60	-	-	-	-	-	-	-	-	-	-
TEST + LL+ 50%WL	11	311	1.20	1.00	-	-	0.80	-	-	-	-	-	-	-
	12	312	1.20	1.00	-	-	-0.80	-	-	-	-	-	-	-
	13	313	1.20	1.00	-	-	-	0.80	-	-	-	-	-	-
	14	314	1.20	1.00	-	-	-	-0.80	-	-	-	-	-	-
ML + LL	15	315	1.20	1.60	1.20	-	-	-	-	-	-	-	-	-
OP+WL+LL	16	316	1.20	1.00	-	1.20	1.60	-	1.20	1.20	-	1.20	1.20	1.20
	17	317	1.20	1.00	-	1.20	-	1.60	1.20	1.20	-	1.20	1.20	1.20
	18	318	1.20	1.00	-	1.20	-1.60	-	-1.20	-1.20	-	1.20	-1.20	-1.20
	19	319	1.20	1.00	-	1.20	-	-1.60	-1.20	-1.20	-	1.20	-1.20	-1.20
0.9OP + WL Without Reserve Load	20	320	0.90	-	-	0.90	1.60	-	0.90	0.90	-	-	-	-
	21	321	0.90	-	-	0.90	-	1.60	0.90	0.90	-	-	-	-
	22	322	0.90	-	-	0.90	-1.60	-	-0.90	-0.90	-	-	-	-
	23	323	0.90	-	-	0.90	-	-1.60	-0.90	-0.90	-	-	-	-
Empty +WL	16	324	0.90	-	0.90	-	1.60	-	-	-	-	-	-	-
	17	325	0.90	-	0.90	-	-1.60	-	-	-	-	-	-	-
	18	326	0.90	-	0.90	-	-	1.60	-	-	-	-	-	-
	19	327	0.90	-	0.90	-	-	-1.60	-	-	-	-	-	-
ERECTION + WL	20	328	0.9 (*)	-	-	-	1.6(**)	-	-	-	-	-	-	-
	21	329	0.9 (*)	-	-	-	-1.6(**)	-	-	-	-	-	-	-
	22	330	0.9 (*)	-	-	-	-	1.6(**)	-	-	-	-	-	-
	23	331	0.9 (*)	-	-	-	-	-1.6(**)	-	-	-	-	-	-
OP+LL+EQ	20	332	1.20	1.00	-	1.20	-	-	1.20	1.20	-	1.20	1.20	1.20
	21	333	1.20	1.00	-	1.20	-	-	1.20	1.20	-	1.20	1.20	1.20
	22	334	1.20	1.00	-	1.20	-	-	-1.20	-1.20	-	1.20	-1.20	-1.20
	23	335	1.20	1.00	-	1.20	-	-	-1.20	-1.20	-	1.20	-1.20	-1.20
0.9OP+1.0 EQ Without Reserve Loads	24	336	0.90	-	-	0.90	-	-	0.90	0.90	-	-	-	-
	25	337	0.90	-	-	0.90	-	-	0.90	0.90	-	-	-	-
	26	338	0.90	-	-	0.90	-	-	-0.90	-0.90	-	-	-	-
	27	339	0.90	-	-	0.90	-	-	-0.90	-0.90	-	-	-	-
1.2OP+1.0 EQ + LL Without Reserve Loads	28	340	1.20	1.00	-	1.20	-	-	1.20	1.20	-	-	-	-
	29	341	1.20	1.00	-	1.20	-	-	1.20	1.20	-	-	-	-
	30	342	1.20	1.00	-	1.20	-	-	-1.20	-1.20	-	-	-	-
	31	343	1.20	1.00	-	1.20	-	-	-1.20	-1.20	-	-	-	-
OP.	32	401	1.40	-	-	1.40	-	-	1.40	1.40	1.40	1.40	1.40	1.40
	33	402	1.40	-	-	1.40	-	-	-1.40	-1.40	-1.40	1.40	-1.40	-1.40
	34	403	1.40	-	-	1.40	-	-	1.40	-1.40	1.40	1.40	1.40	-1.40
	35	404	1.40	-	-	1.40	-	-	-1.40	1.40	-1.40	1.40	-1.40	1.40
OP. + LL	36	405	1.20	1.60	-	1.20	-	-	1.20	1.20	1.20	1.20	1.20	1.20
	37	406	1.20	1.60	-	1.20	-	-	-1.20	-1.20	-1.20	1.20	-1.20	-1.20
	38	407	1.20	1.60	-	1.20	-	-	-1.20	1.20	-1.20	1.20	-1.20	1.20
	39	408	1.20	1.60	-	1.20	-	-	1.20	-1.20	1.20	1.20	1.20	-1.20
OP+WL+LL	40	409	1.20	1.00	-	1.20	1.60	-	1.20	1.20	1.20	1.20	1.20	1.20
	41	410	1.20	1.00	-	1.20	-	1.60	1.20	1.20	1.20	1.20	1.20	1.20
	42	411	1.20	1.00	-	1.20	-1.60	-	-1.20	-1.20	-1.20	1.20	-1.20	-1.20
	43	412	1.20	1.00	-	1.20	-	-1.60	-1.20	-1.20	-1.20	1.20	-1.20	-1.20
TEST + LL+ 50%WL	44	413	1.20	1.00	-	-	0.80	-	-	-	-	-	-	-
	45	414	1.20	1.00	-	-	-0.80	-	-	-	-	-	-	-
	46	415	1.20	1.00	-	-	-	0.80	-	-	-	-	-	-
	47	416	1.20	1.00	-	-	-	-0.80	-	-	-	-	-	-
OP. + PSV	48	417	1.40	-	-	1.40	-	-	1.40	1.40	1.40	1.40	1.40	1.40
	49	418	1.40	-	-	1.40	-	-	-1.40	-1.40	-1.40	1.40	-1.40	-1.40
	50	419	1.40	-	-	1.40	-	-	1.40	-1.40	1.40	1.40	1.40	-1.40
	51	420	1.40	-	-	1.40	-	-	-1.40	1.40	-1.40	1.40	-1.40	1.40
OP. + LL + PSV	52	421	1.20	1.60	-	1.20	-	-	1.20	1.20	1.20	1.20	1.20	1.20
	53	422	1.20	1.60	-	1.20	-	-	-1.20	-1.20	-1.20	1.20	-1.20	-1.20
	54	423	1.20	1.60	-	1.20	-	-	-1.20	1.20	-1.20	1.20	-1.20	1.20
	55	424	1.20	1.60	-	1.20	-	-	1.20	-1.20	1.20	1.20	1.20	-1.20

LOAD COMBINATIONS AS PER ASCE 7-05 (CHAPTER 7)

Load Conditions	Sr. No.	Load Comb	1 DL	2 LL	3 DL_{empty}	4 DL_{op}	6 WLX	7 WLZ	10 TL_X	11 TL_Z	12 $TL_{Min}X$	50 R_V	51 R_{HE}	52 R_{HN}
OP.	1		-	-	-	-	-	-	-	-	-	-	-	-
	2		-	-	-	-	-	-	-	-	-	-	-	-
	3		-	-	-	-	-	-	-	-	-	-	-	-
	4		-	-	-	-	-	-	-	-	-	-	-	-
OP. + LL	5	601	1.00	1.00	-	1.00	-	-	1.00	1.00	-	1.00	1.00	1.00
	6	602	1.00	1.00	-	1.00	-	-	-1.00	-1.00	-	1.00	-1.00	-1.00
	7	603	1.00	1.00	-	1.00	-	-	1.00	-1.00	-	1.00	1.00	-1.00
	8	604	1.00	1.00	-	1.00	-	-	-1.00	1.00	-	1.00	-1.00	1.00
Test	9		-	-	-	-	-	-	-	-	-	-	-	-
Test+ LL	10	605	1.00	1.00	-	-	-	-	-	-	-	-	-	-
TEST + LL+ 50%WL	11	606	1.00	0.75	-	-	0.38	-	-	-	-	-	-	-
	12	607	1.00	0.75	-	-	-0.38	-	-	-	-	-	-	-
	13	608	1.00	0.75	-	-	-	0.38	-	-	-	-	-	-
	14	609	1.00	0.75	-	-	-	-0.38	-	-	-	-	-	-
OP.	32		-	-	-	-	-	-	-	-	-	-	-	-
	33		-	-	-	-	-	-	-	-	-	-	-	-
	34		-	-	-	-	-	-	-	-	-	-	-	-
	35		-	-	-	-	-	-	-	-	-	-	-	-
OP. + LL	36	701	1.00	1.00	-	1.00	-	-	1.00	1.00	1.00	1.00	1.00	1.00
	37	702	1.00	1.00	-	1.00	-	-	-1.00	-1.00	-1.00	1.00	-1.00	-1.00
	38	703	1.00	1.00	-	1.00	-	-	1.00	-1.00	1.00	1.00	1.00	-1.00
	39	704	1.00	1.00	-	1.00	-	-	-1.00	1.00	-1.00	1.00	-1.00	1.00
TEST + LL+ 50%WL	44	705	1.00	0.75	-	-	0.38	-	-	-	-	-	-	-
	45	706	1.00	0.75	-	-	-0.38	-	-	-	-	-	-	-
	46	707	1.00	0.75	-	-	-	0.38	-	-	-	-	-	-
	47	708	1.00	0.75	-	-	-	-0.38	-	-	-	-	-	-
OP+WL		801	1.00		-	1.00	1.00	-	1.00	1.00	-	1.00	1.00	1.00
		802	1.00		-	1.00	-	1.00	1.00	1.00	-	1.00	1.00	1.00
		803	1.00		-	1.00	-1.00	-	-1.00	-1.00	-	1.00	-1.00	-1.00
		804	1.00		-	1.00	-	-1.00	-1.00	-1.00	-	1.00	-1.00	-1.00
OP+WL Without Reserve Load	16	805	1.00		-	1.00	1.00	-	1.00	1.00	-			
	17	806	1.00		-	1.00	-	1.00	1.00	1.00	-			
	18	807	1.00		-	1.00	-1.00	-	-1.00	-1.00	-			
	19	808	1.00		-	1.00	-	-1.00	-1.00	-1.00	-			
OP+WL+L L	16	809	1.00	0.75	-	1.00	0.75	-	1.00	1.00	-	1.00	1.00	1.00
	17	810	1.00	0.75	-	1.00	-	0.75	1.00	1.00	-	1.00	1.00	1.00
	18	811	1.00	0.75	-	1.00	-0.75	-	-1.00	-1.00	-	1.00	-1.00	-1.00
	19	812	1.00	0.75	-	1.00	-	-0.75	-1.00	-1.00	-	1.00	-1.00	-1.00
0.6OP + WL Without Reserve Load	20	813	0.60	-	-	0.60	1.00	-	0.60	0.60	-	-	-	-
	21	814	0.60	-	-	0.60	-	1.00	0.60	0.60	-	-	-	-
	22	815	0.60	-	-	0.60	-1.00	-	-0.60	-0.60	-	-	-	-
	23	816	0.60	-	-	0.60	-	-1.00	-0.60	-0.60	-	-	-	-
Empty +WL	16	817	0.60	-	0.60	-	1.00	-	-	-	-	-	-	-
	17	818	0.60	-	0.60	-	-1.00	-	-	-	-	-	-	-
	18	819	0.60	-	0.60	-	-	1.00	-	-	-	-	-	-
	19	820	0.60	-	0.60	-	-	-1.00	-	-	-	-	-	-
ERECTION + WL	20	821	0.6 (*)	-	-	-	1.0 (**)	-	-	-	-	-	-	-
	21	822	0.6 (*)	-	-	-	-1.0(**)	-	-	-	-	-	-	-
	22	823	0.6 (*)	-	-	-	-	1.0 (**)	-	-	-	-	-	-
	23	824	0.6 (*)	-	-	-	-	-1.0(**)	-	-	-	-	-	-
TEST + 50%WL		825	1.00		-	-	0.50	-	-	-	-	-	-	-
		826	1.00		-	-	-0.50	-	-	-	-	-	-	-
		827	1.00		-	-	-	0.50	-	-	-	-	-	-
		828	1.00		-	-	-	-0.50	-	-	-	-	-	-
OP+EQ		829	1.00		-	1.00	-	-	1.00	1.00	-	1.00	1.00	1.00
		830	1.00		-	1.00	-	-	1.00	1.00	-	1.00	1.00	1.00
		831	1.00		-	1.00	-	-	-1.00	-1.00	-	1.00	-1.00	-1.00
		832	1.00		-	1.00	-	-	-1.00	-1.00	-	1.00	-1.00	-1.00
OP+LL+EQ	20	833	1.00	0.75	-	1.00	-	-	1.00	1.00	-	1.00	1.00	1.00
	21	834	1.00	0.75	-	1.00	-	-	1.00	1.00	-	1.00	1.00	1.00
	22	835	1.00	0.75	-	1.00	-	-	-1.00	-1.00	-	1.00	-1.00	-1.00
	23	836	1.00	0.75	-	1.00	-	-	-1.00	-1.00	-	1.00	-1.00	-1.00
0.6OP+EQ Without Reserve Loads	24	837	0.60	-	-	0.60	-	-	0.60	0.60	-	-	-	-
	25	838	0.60	-	-	0.60	-	-	0.60	0.60	-	-	-	-
	26	839	0.60	-	-	0.60	-	-	-0.60	-0.60	-	-	-	-
	27	840	0.60	-	-	0.60	-	-	-0.60	-0.60	-	-	-	-
OP+EQ Without Reserve Loads	28	841	1.00		-	1.00	-	-	1.00	1.00	-	-	-	-
	29	842	1.00		-	1.00	-	-	1.00	1.00	-	-	-	-
	30	843	1.00		-	1.00	-	-	-1.00	-1.00	-	-	-	-
	31	844	1.00		-	1.00	-	-	-1.00	-1.00	-	-	-	-
M L+ LL	15	845	1.00	1.00	1.00	-	-	-	-	-	-	-	-	-

LOAD COMBINATIONS FOR LIMIT STATE DESIGN AS PER IS:800 2007(Chapter 8)

Load Conditions	Sr. No.	Load Comb	1 DL	2 LL	3 DL_{empty}	4 DL_{op}	6 WLE	7 WLN	10 TL_E	11 TL_N	50 R_V	51 R_{HN}	52 R_{HE}
OP. + LL	1	301	1.50	1.50	-	1.50	-	-	1.50	1.50	1.50	1.50	1.50
	2	302	1.50	1.50	-	1.50	-	-	-1.50	-1.50	1.50	-1.50	-1.50
	3	303	1.50	1.50	-	1.50	-	-	-1.50	1.50	1.50	-1.50	1.50
	4	304	1.50	1.50	-	1.50	-	-	1.50	-1.50	1.50	1.50	-1.50
OP+WL+LL	5	305	1.20	1.20	-	1.20	1.20	-	1.20	1.20	1.20	1.20	1.20
	6	306	1.20	1.20	-	1.20	-	1.20	1.20	1.20	1.20	1.20	1.20
	7	307	1.20	1.20	-	1.20	-1.20	-	-1.20	-1.20	1.20	-1.20	-1.20
	8	308	1.20	1.20	-	1.20	-	-1.20	-1.20	-1.20	1.20	-1.20	-1.20
OP + WL	9	309	1.50	-	-	1.50	1.50	-	1.50	1.50	1.50	1.50	1.50
	10	310	1.50	-	-	1.50	-	1.50	1.50	1.50	1.50	1.50	1.50
	11	311	1.50	-	-	1.50	-1.50	-	-1.50	-1.50	1.50	-1.50	-1.50
	12	312	1.50	-	-	1.50	-	-1.50	-1.50	-1.50	1.50	-1.50	-1.50
OP + WL w/o Reserve load	13	313	1.50	-	-	1.50	1.50	-	1.50	1.50	-	-	-
	14	314	1.50	-	-	1.50	-	1.50	1.50	1.50	-	-	-
	15	315	1.50	-	-	1.50	-1.50	-	-1.50	-1.50	-	-	-
	16	316	1.50	-	-	1.50	-	-1.50	-1.50	-1.50	-	-	-
OP+LL+EQ	17	317	1.20	1.20	-	1.20	-	-	1.20	1.20	1.20	1.20	1.20
	18	318	1.20	1.20	-	1.20	-	-	1.20	1.20	1.20	1.20	1.20
	19	319	1.20	1.20	-	1.20	-	-	-1.20	-1.20	1.20	-1.20	-1.20
	20	320	1.20	1.20	-	1.20	-	-	-1.20	-1.20	1.20	-1.20	-1.20
OP + EQ	21	321	1.50	-	-	1.50			1.50	1.50	1.50	1.50	1.50
	22	322	1.50	-	-	1.50			1.50	1.50	1.50	1.50	1.50
	23	323	1.50	-	-	1.50			-1.50	-1.50	1.50	-1.50	-1.50
	24	324	1.50	-	-	1.50			-1.50	-1.50	1.50	-1.50	-1.50
OP + EQ w/o Reserve load	25	325	1.50	-	-	1.50			1.50	1.50	-	-	-
	26	326	1.50	-	-	1.50			1.50	1.50	-	-	-
	27	327	1.50	-	-	1.50			-1.50	-1.50	-	-	-
	28	328	1.50	-	-	1.50			-1.50	-1.50	-	-	-
Empty + WL	29	329	0.90	-	0.90	-	1.50	-	-	-	-	-	-
	30	330	0.90	-	0.90	-	-	1.50	-	-	-	-	-
	31	331	0.90	-	0.90	-	-1.50	-	-	-	-	-	-
	32	332	0.90	-	0.90	-	-	-1.50	-	-	-	-	-
TEST + LL	33	333	1.50	1.50	-	-	-	-	-	-	-	-	-
TEST + LL+ 25%WL	34	334	1.20	1.20	-	-	0.30	-	-	-	-	-	-
	35	335	1.20	1.20	-	-	-	0.30	-	-	-	-	-
	36	336	1.20	1.20	-	-	-0.30	-	-	-	-	-	-
	37	337	1.20	1.20	-	-	-	-0.30	-	-	-	-	-
ML+LL	38	338	1.50	1.50	1.50	-	-	-	-	-	-	-	-
OP. + LL	39	401	1.50	1.50	-	1.50	-	-	1.50	1.50	1.50	1.50	1.50
	40	402	1.50	1.50	-	1.50	-	-	-1.50	-1.50	1.50	-1.50	-1.50
	41	403	1.50	1.50	-	1.50	-	-	-1.50	1.50	1.50	-1.50	1.50
	42	404	1.50	1.50	-	1.50	-	-	1.50	-1.50	1.50	1.50	-1.50
OP+WL+LL	43	405	1.20	1.20	-	1.20	1.20	-	1.20	1.20	1.20	1.20	1.20
	44	406	1.20	1.20	-	1.20	-	1.20	1.20	1.20	1.20	1.20	1.20
	45	407	1.20	1.20	-	1.20	-1.20	-	-1.20	-1.20	1.20	-1.20	-1.20
	46	408	1.20	1.20	-	1.20	-	-1.20	-1.20	-1.20	1.20	-1.20	-1.20
OP. + WL	47	409	1.50	-	-	1.50	1.50	-	1.50	1.50	1.50	1.50	1.50
	48	410	1.50	-	-	1.50	-	1.50	1.50	1.50	1.50	1.50	1.50
	49	411	1.50	-	-	1.50	-1.50	-	-1.50	-1.50	1.50	-1.50	-1.50
	50	412	1.50	-	-	1.50	-	-1.50	-1.50	-1.50	1.50	-1.50	-1.50
OP+EQ+LL	51	413	1.20	1.20	-	1.20	-	-	1.20	1.20	1.20	1.20	1.20
	52	414	1.20	1.20	-	1.20	-	-	1.20	1.20	1.20	1.20	1.20
	53	415	1.20	1.20	-	1.20	-	-	-1.20	-1.20	1.20	-1.20	-1.20
	54	416	1.20	1.20	-	1.20	-	-	-1.20	-1.20	1.20	-1.20	-1.20
OP + EQ	55	417	1.50	-	-	1.50			1.50	1.50	1.50	1.50	1.50
	56	418	1.50	-	-	1.50			1.50	1.50	1.50	1.50	1.50
	57	419	1.50	-	-	1.50			-1.50	-1.50	1.50	-1.50	-1.50
	58	420	1.50	-	-	1.50			-1.50	-1.50	1.50	-1.50	-1.50
OP. + PSV	59	421	1.50	-	-	1.50	-	-	1.50	1.50	1.50	1.50	1.50
	60	422	1.50	-	-	1.50	-	-	-1.50	-1.50	1.50	-1.50	-1.50
	61	423	1.50	-	-	1.50	-	-	1.50	-1.50	1.50	1.50	-1.50
	62	424	1.50	-	-	1.50	-	-	-1.50	1.50	1.50	-1.50	1.50
OP. + LL + PSV	63	425	1.50	1.50	-	1.50	-	-	1.50	1.50	1.50	1.50	1.50
	64	426	1.50	1.50	-	1.50	-	-	-1.50	-1.50	1.50	-1.50	-1.50
	65	427	1.50	1.50	-	1.50	-	-	1.50	-1.50	1.50	1.50	-1.50
	66	428	1.50	1.50	-	1.50	-	-	-1.50	1.50	1.50	-1.50	1.50

"LOAD COMBINATIONS FOR AS PER IS:800 2007(Chapter 8)"

Load	Sr.	Load	1	2	3	4	6	7	10	11	50	51	52
Conditions	No.	Comb	DL	LL	DL_{empty}	DL_{op}	WLE	WLN	TL_E	TL_N	R_V	R_{HN}	R_{HE}
OP. + LL	1	601	1.00	1.00	-	1.00	-	-	1.00	1.00	1.00	1.00	1.00
	2	602	1.00	1.00	-	1.00	-	-	-1.00	-1.00	1.00	-1.00	-1.00
	3	603	1.00	1.00	-	1.00	-	-	-1.00	1.00	1.00	-1.00	1.00
	4	604	1.00	1.00	-	1.00	-	-	1.00	-1.00	1.00	1.00	-1.00
OP+WL+LL	5	605	1.00	0.80	-	1.00	0.80	-	1.00	1.00	1.00	1.00	1.00
	6	606	1.00	0.80	-	1.00	-	0.80	1.00	1.00	1.00	1.00	1.00
	7	607	1.00	0.80	-	1.00	-0.80	-	-1.00	-1.00	1.00	-1.00	-1.00
	8	608	1.00	0.80	-	1.00	-	-0.80	-1.00	-1.00	1.00	-1.00	-1.00
OP + WL	9	609	1.00	-	-	1.00	1.00	-	1.00	1.00	1.00	1.00	1.00
	10	610	1.00	-	-	1.00	-	1.00	1.00	1.00	1.00	1.00	1.00
	11	611	1.00	-	-	1.00	-1.00	-	-1.00	-1.00	1.00	-1.00	-1.00
	12	612	1.00	-	-	1.00	-	-1.00	-1.00	-1.00	1.00	-1.00	-1.00
OP + WL w/o Reserve load	13	613	1.00	-	-	1.00	1.00	-	1.00	1.00	-	-	-
	14	614	1.00	-	-	1.00	-	1.00	1.00	1.00	-	-	-
	15	615	1.00	-	-	1.00	-1.00	-	-1.00	-1.00	-	-	-
	16	616	1.00	-	-	1.00	-	-1.00	-1.00	-1.00	-	-	-
OP+LL+EQ	17	617	1.00	0.80	-	1.00	-	-	1.00	1.00	1.00	1.00	1.00
	18	618	1.00	0.80	-	1.00	-	-	1.00	1.00	1.00	1.00	1.00
	19	619	1.00	0.80	-	1.00	-	-	-1.00	-1.00	1.00	-1.00	-1.00
	20	620	1.00	0.80	-	1.00	-	-	-1.00	-1.00	1.00	-1.00	-1.00
OP + EQ	21	621	1.00	-	-	1.00			1.00	1.00	1.00	1.00	1.00
	22	622	1.00	-	-	1.00			1.00	1.00	1.00	1.00	1.00
	23	623	1.00	-	-	1.00			-1.00	-1.00	1.00	-1.00	-1.00
	24	624	1.00	-	-	1.00			-1.00	-1.00	1.00	-1.00	-1.00
OP + EQ w/o Reserve load	25	625	1.00	-	-	1.00			1.00	1.00	-	-	-
	26	626	1.00	-	-	1.00			1.00	1.00	-	-	-
	27	627	1.00	-	-	1.00			-1.00	-1.00	-	-	-
	28	628	1.00	-	-	1.00			-1.00	-1.00	-	-	-
Empty + WL	29	629	1.00	-	1.00	-	1.00	-	-	-	-	-	-
	30	630	1.00	-	1.00	-	-	1.00	-	-	-	-	-
	31	631	1.00	-	1.00	-	-1.00	-	-	-	-	-	-
	32	632	1.00	-	1.00	-	-	-1.00	-	-	-	-	-
TEST + LL	33	633	1.00	1.00	-	-	-	-	-	-	-	-	-
TEST + LL+ 25%WL	34	634	1.00	0.80	-	-	0.20	-	-	-	-	-	-
	35	635	1.00	0.80	-	-	-	0.20	-	-	-	-	-
	36	636	1.00	0.80	-	-	-0.20	-	-	-	-	-	-
	37	637	1.00	0.80	-	-	-	-0.20	-	-	-	-	-
ML+LL	38	638	1.00	1.00	1.00	-	-	-	-	-	-	-	-
OP. + LL	39	701	1.00	1.00	-	1.00	-	-	1.00	1.00	1.00	1.00	1.00
	40	702	1.00	1.00	-	1.00	-	-	-1.00	-1.00	1.00	-1.00	-1.00
	41	703	1.00	1.00	-	1.00	-	-	-1.00	1.00	1.00	-1.00	1.00
	42	704	1.00	1.00	-	1.00	-	-	1.00	-1.00	1.00	1.00	-1.00

LOAD COMBINATIONS FOR WORKING STRESS DESIGN AS PER IS:800 1984(Chapter 8)

Load Conditions	Sr. No.	Load Comb	1 DL	2 LL	3 DL_{empty}	4 DL_{op}	6 WLE	7 WLN	10 TL_E	11 TL_N	50 R_V	51 R_{HN}	52 R_{HE}
OP. + LL	1	301	1.00	1.00	-	1.00	-	-	1.00	1.00	1.00	1.00	1.00
	2	302	1.00	1.00	-	1.00	-	-	-1.00	-1.00	1.00	-1.00	-1.00
	3	303	1.00	1.00	-	1.00	-	-	-1.00	1.00	1.00	-1.00	1.00
	4	304	1.00	1.00	-	1.00	-	-	1.00	-1.00	1.00	1.00	-1.00
OP+WL+LL	5	305	1.00	1.00	-	1.00	1.00	-	1.00	1.00	1.00	1.00	1.00
	6	306	1.00	1.00	-	1.00	-	1.00	1.00	1.00	1.00	1.00	1.00
	7	307	1.00	1.00	-	1.00	-1.00	-	-1.00	-1.00	1.00	-1.00	-1.00
	8	308	1.00	1.00	-	1.00	-	-1.00	-1.00	-1.00	1.00	-1.00	-1.00
OP + WL	9	309	1.00	-	-	1.00	1.00	-	1.00	1.00	1.00	1.00	1.00
	10	310	1.00	-	-	1.00	-	1.00	1.00	1.00	1.00	1.00	1.00
	11	311	1.00	-	-	1.00	-1.00	-	-1.00	-1.00	1.00	-1.00	-1.00
	12	312	1.00	-	-	1.00	-	-1.00	-1.00	-1.00	1.00	-1.00	-1.00
OP + WL w/o Reserve load	13	313	1.00	-	-	1.00	1.00	-	1.00	1.00	-	-	-
	14	314	1.00	-	-	1.00	-	1.00	1.00	1.00	-	-	-
	15	315	1.00	-	-	1.00	-1.00	-	-1.00	-1.00	-	-	-
	16	316	1.00	-	-	1.00	-	-1.00	-1.00	-1.00	-	-	-
OP+LL+EQ	17	317	1.00	1.00	-	1.00	-	-	1.00	1.00	1.00	1.00	1.00
	18	318	1.00	1.00	-	1.00	-	-	1.00	1.00	1.00	1.00	1.00
	19	319	1.00	1.00	-	1.00	-	-	-1.00	-1.00	1.00	-1.00	-1.00
	20	320	1.00	1.00	-	1.00	-	-	-1.00	-1.00	1.00	-1.00	-1.00
OP + EQ	21	321	1.00	-	-	1.00			1.00	1.00	1.00	1.00	1.00
	22	322	1.00	-	-	1.00			1.00	1.00	1.00	1.00	1.00
	23	323	1.00	-	-	1.00			-1.00	-1.00	1.00	-1.00	-1.00
	24	324	1.00	-	-	1.00			-1.00	-1.00	1.00	-1.00	-1.00
OP + EQ w/o Reserve load	25	325	1.00	-	-	1.00			1.00	1.00	-	-	-
	26	326	1.00	-	-	1.00			1.00	1.00	-	-	-
	27	327	1.00	-	-	1.00			-1.00	-1.00	-	-	-
	28	328	1.00	-	-	1.00			-1.00	-1.00	-	-	-
Empty + WL	29	329	0.90	-	0.90	-	1.00	-	-	-	-	-	-
	30	330	0.90	-	0.90	-	-	1.00	-	-	-	-	-
	31	331	0.90	-	0.90	-	-1.00	-	-	-	-	-	-
	32	332	0.90	-	0.90	-	-	-1.00	-	-	-	-	-
TEST + LL	33	333	1.00	1.00	-	-	-	-	-	-	-	-	-
TEST + LL+ 25%WL	34	334	1.00	1.00	-	-	0.30	-	-	-	-	-	-
	35	335	1.00	1.00	-	-	-	0.30	-	-	-	-	-
	36	336	1.00	1.00	-	-	-0.30	-	-	-	-	-	-
	37	337	1.00	1.00	-	-	-	-0.30	-	-	-	-	-
ML+LL	38	338	1.00	1.00	1.00	-	-	-	-	-	-	-	-
OP. + LL	39	401	1.00	1.00	-	1.00	-	-	1.00	1.00	1.00	1.00	1.00
	40	402	1.00	1.00	-	1.00	-	-	-1.00	-1.00	1.00	-1.00	-1.00
	41	403	1.00	1.00	-	1.00	-	-	-1.00	1.00	1.00	-1.00	1.00
	42	404	1.00	1.00	-	1.00	-	-	1.00	-1.00	1.00	1.00	-1.00
OP+WL+LL	43	405	1.00	1.00	-	1.00	1.00	-	1.00	1.00	1.00	1.00	1.00
	44	406	1.00	1.00	-	1.00	-	1.00	1.00	1.00	1.00	1.00	1.00
	45	407	1.00	1.00	-	1.00	-1.00	-	-1.00	-1.00	1.00	-1.00	-1.00
	46	408	1.00	1.00	-	1.00	-	-1.00	-1.00	-1.00	1.00	-1.00	-1.00
OP. + WL	47	409	1.00	-	-	1.00	1.00	-	1.00	1.00	1.00	1.00	1.00
	48	410	1.00	-	-	1.00	-	1.00	1.00	1.00	1.00	1.00	1.00
	49	411	1.00	-	-	1.00	-1.00	-	-1.00	-1.00	1.00	-1.00	-1.00
	50	412	1.00	-	-	1.00	-	-1.00	-1.00	-1.00	1.00	-1.00	-1.00
OP+EQ+LL	51	413	1.00	1.00	-	1.00	-	-	1.00	1.00	1.00	1.00	1.00
	52	414	1.00	1.00	-	1.00	-	-	1.00	1.00	1.00	1.00	1.00
	53	415	1.00	1.00	-	1.00	-	-	-1.00	-1.00	1.00	-1.00	-1.00
	54	416	1.00	1.00	-	1.00	-	-	-1.00	-1.00	1.00	-1.00	-1.00
OP + EQ	55	417	1.00	-	-	1.00			1.00	1.00	1.00	1.00	1.00
	56	418	1.00	-	-	1.00			1.00	1.00	1.00	1.00	1.00
	57	419	1.00	-	-	1.00			-1.00	-1.00	1.00	-1.00	-1.00
	58	420	1.00	-	-	1.00			-1.00	-1.00	1.00	-1.00	-1.00
OP. + PSV	59	421	1.00	-	-	1.00	-	-	1.00	1.00	1.00	1.00	1.00
	60	422	1.00	-	-	1.00	-	-	-1.00	-1.00	1.00	-1.00	-1.00
	61	423	1.00	-	-	1.00	-	-	1.00	-1.00	1.00	1.00	-1.00
	62	424	1.00	-	-	1.00	-	-	-1.00	1.00	1.00	-1.00	1.00
OP. + LL + PSV	63	425	1.00	1.00	-	1.00	-	-	1.00	1.00	1.00	1.00	1.00
	64	426	1.00	1.00	-	1.00	-	-	-1.00	-1.00	1.00	-1.00	-1.00
	65	427	1.00	1.00	-	1.00	-	-	1.00	-1.00	1.00	1.00	-1.00
	66	428	1.00	1.00	-	1.00	-	-	-1.00	1.00	1.00	-1.00	1.00

Appendix B

Load Calculation

B.1 Second order analysis using moment amplification Using IS:800 2007

Axial force	P =	603	kN	
Moment	***M***	$\boldsymbol{M_z}$	$\boldsymbol{M_y}$	
	at Base	0	0	*kNm*
	at top	499	0	*kNm*
	f_y =	345	N/mm^2	
	E =	200000	N/mm^2	
Length of column	L =	6	m	
Effective Length @ y	Kl_y =	6	m	
Effective Length @ z	Kl_z =	14.16	m	
Partial safety factor	γ_{mo} =	1.1		
Plastic section modulus	Z_{pz} =	3458044.57	mm^3	
	Z_{py} =	1685618.51	mm^3	

Try W 14X120 ***OK***

Weight per Meter (w)	W =	179	kg/m
Sectional Area (a)	A =	22774	mm^2
Depth of Section (h)	h =	368	mm
Width of flange (b)	b_f =	374	mm
Thickness of Flange (t_f)	t_f =	24	mm
Thickness of Web (t_w)	t_w =	15	mm
Moment of Inertia	I_{xx} =	574399400	mm^4
	I_{yy} =	206034500	mm^4
Raddi of Gyration	r_{xx} =	159	mm
	r_{yy} =	95	mm
Modulli of Section	Z_{xx} =	3113540	mm^3
	Z_{yy} =	1106130	mm^3
Radius at root	R_1 =	20	mm

(1) Type of section: **clause 3.7**

ε = 0.85

outstand flange b/t_f = 7.83

IS:800-2007 table 2, page no-18

for flanges *IS:800-2007 table 2 Page No.-18*

Limit for Class 1	8.00
Limit for Class 2	8.94
Limit for Class 3	13.36

Flenges are Plastic

$d = h-(2\,t_f)-(2\,R_1)$ = 280.58

d/t_w = 18.73

for web

Web is Plastic

Cross-section is Plastic

2. Check for resistance of cross-section to the combined effects fro yielding

Clause(9.3.1.2)

Factored applied axial force	N =	603	kN	*IS:800-2007*
Design strength	N_d =	7142.75	kN	*clause 9.3.1.1*
Design strength in Bending	M_{dy} =	528.67	kN m	*Page No-70*
	M_{dz} =	1084.57	kN m	
	$n = (N / N_d)$ =	0.08		
Design reduced flexural strength	M_{ndy} =	528.67	kN m	
	M_{ndz} =	1084.57	<= Md_z	

IS:800-2007 clause 9.3.1.1,Table-17, page no-71

Constants	α_1 =		1
	α_2 =		2

for plastic and compact section....**1**

$$\left(\frac{M_y}{M_{ndy}}\right)^{\alpha_1}+\left(\frac{M_z}{M_{ndz}}\right)^{\alpha_2}\leq 1.0$$

Interaction equation = 0.21

<= 1

Section is O.K.

Section is plastic so equation 1 is consider for design.

For semi compact section........**2**

$$\frac{N}{N_d}+\frac{M_y}{M_{ndy}}+\frac{M_z}{M_{ndz}}\leq 1.0$$

Interaction equation = 0.5

<= 1

Section is O.K.

3. Buckling resistance in compression

clause(7.1.2)

KL_z/r_{zz} = 89.34

KL_y/r_{yy} = 63.16

$\lambda_1 = \pi\,(E/f_y)^{0.5}$ = 75.67

Non dimensional effective slenderness ratio — *IS:800-2007*

$\lambda_z = (\lambda/\lambda_1)$ =	1.18		*clause 7.1.2.1*
$\lambda_y = (\lambda/\lambda_1)$ =	0.83		*Page No-34*
h/b_f =	0.99		
t_f =	23.87	mm	
Imperfaction factor α_z =	0.34	b	
α_y =	0.49	c	

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

ϕ_z = 1.36

ϕ_y = 1.00

Stress reduction factor

$$\chi = \frac{1}{\phi+(\phi^2-\bar{\lambda}^2)^{\frac{1}{2}}}\leq 1.0$$

χ_z = 0.49 ≤ 1

χ_y = 0.64 ≤ 1

Design Compressive strength — *IS:800-2007*

$P_d = (A_e \; X \; f_y)/\gamma_{mo}$ — *clause 7.1.2*

P_{dz} = 3491.12 kN — *Page No-34*

P_{dy} = 4574.42 kN

Section is O.K.

4. Shear resistance of cross section

clause(8.4.1)

Design plastic shear resistance	$V_p = A_v(f_y/(3)^{0.5})/\gamma_{mo}$		
Load parallel to web due to moment @ **major axis**			
Design plastic shear resistance	$V_p =$ 999.03		
Maximum shear force on section	$V =$ 83.17		
Load parallel to flanges due to moment @ **minor axis**			
Design plastic shear resistance	$V_p =$ 3231.38		
Maximum shear force on section	$V =$ 0.00		*IS:800-2007*
			clause 8.4
	Section is O.K.		*Page No-59*
Shear buckling clause(8.4.2)			
	$d/t_w =$ 21.40	$> 67\varepsilon$	*IS:800-2007*
IS:800-2007 clause 8.4.2, page no-59			*clause 8.4.2*
	Section is O.K.		*Page No-59*

5. Buckling resistance in bending

(clause 8.2.2)

IS:800-2007 clause 8.2.2.1, page no-54

Elastic lateral buckling moment

$$M_{cr} = \frac{\pi^2 E I_y h_f}{2 L_{LT}^2}\left[1+\frac{1}{20}\left[\frac{L_{LT}/r_y}{h_f/t_f}\right]^2\right]^{0.5}$$

$M_{cr} =$ 2822503294 N mm

$$f_{crb} = \frac{1.1\pi^2 E}{(L_{LT}/r_y)^2}\left[1+\frac{1}{20}\left[\frac{L_{LT}/r_y}{h_f/t_f}\right]^2\right]^{0.5}$$

$f_{crb} =$ 850.89

Non dimensional slenderness ratio

$$\lambda_{LT} = \sqrt{Z_{ez} \cdot f_y / M_{cr}}$$

$\lambda_{LT} = 0.64$

Imperfection parameter $\alpha_{LT} =$ 0.21 *Page No-54*

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2\right]$$

$\phi_{LT} =$ 0.75

Bending stress reduction factor

$$\chi_{LT} = \left[\frac{1}{\phi_{LT} + \left\{\phi_{LT}^2 - \lambda_{LT}^2\right\}^{0.5}}\right] \le 1.0$$

$X_{LT} =$ 0.88 ≤ 1

Design bending compressive stress

$$f_{bd} = \chi_{LT}\, f_y / \gamma_m$$

$f_{bd} =$ 274.59 N/mm^2

Lateral torsional buckling resistance

$$M_d = Z_e f_{bd}$$

$M_{dz} =$ 949.55 kN m

$M_{dy} =$ 528.67 kN m

Section is O.K.

6. Buckling resistance in bending & axial comp.
(clause 9.3.2.2)

$\psi_z = M_2/M_1 =$ 0.00

$\psi_y = M_2/M_1 =$ 0.00

Equivalent uniform moment factor

$C_{mz} = 0.6 + 0.4\,\psi \geq 0.4$	$C_{mz} =$	0.60	≥ 0.4
$C_{my} = 0.6 + 0.4\,\psi \geq 0.4$	$C_{my} =$	0.60	≥ 0.4

Equivalent uniform moment factor for lateral torsional buckling

$C_{mLT} = 0.6 + 0.4\,\psi \geq 0.4$	$C_{mLT} =$	0.60	≥ 0.4
n_y	$K_y =$	1.08	
$k_z = 1+(\lambda_z-0.2)n_z \leq 1+0.8\,n_z$	$K_z =$	1.14	
	$n_y =$	0.13	
	$n_z =$	0.17	
	$K_{LT} =$	0.98	

Check No : 1	$\dfrac{P}{P_{dy}}+k_y\dfrac{C_{my}M_y}{M_{dy}}+k_{LT}\dfrac{M_z}{M_{dz}} \leq 1.0$	**0.64**	<=	1
Check No : 2	$\dfrac{P}{P_{dz}}+0.6k_y\dfrac{C_{my}M_y}{M_{dy}}+k_z\dfrac{C_{mz}M_z}{M_{dz}} \leq 1.0$	**0.53**	<=	1

Section is O.K.

B.2 Advanced analysis using Appendix B Using IS:800 2007

Axial force	P =	617	kN		
Moment	***M***	M_z	M_y		
	at Base	0	0	*kNm*	
	at top	527	0	*kNm*	
	f_y =	345	N/mm^2		
	E =	200000	N/mm^2		
Length of column	L =	6	m		
Effective Length @ y	Kl_y =	6	m		
Effective Length @ z	Kl_z =	6	m		
Partial safety factor	γ_{mo} =	1.1			
Plastic section modulus	Z_{pz} =	3458044.57	mm^3		
	Z_{py} =	1685618.51	mm^3		
Try W 14X120	***OK***				
Weight per Meter (w)	W =	179	kg/m		
Sectional Area (a)	A =	22774	mm^2		
Depth of Section (h)	h =	368	mm		
Width of flange (b)	b_f =	374	mm		
Thickness of Flange (t_f)	t_f =	24	mm		
Thickness of Web (t_w)	t_w =	15	mm		
Moment of Inertia	I_{xx} =	574399400	mm^4		
	I_{yy} =	206034500	mm^4		
Raddi of Gyration	r_{xx} =	159	mm		
	r_{yy} =	95	mm		
Modulli of Section	Z_{xx} =	3113540	mm^3		
	Z_{yy} =	1106130	mm^3		
Radius at root	R_1 =	20	mm		
(1) Type of section:					**clause 3.7**
	ε =	0.85			
outstand flange	b/t_f =	7.83			
IS:800-2007 table 2, page no-18					
for flanges					*IS:800-2007*
Limit for Class 1	8.00				*table 2*
Limit for Class 2	8.94				*Page No.-18*
Limit for Class 3	13.36				
Flenges are	**Plastic**				
	$d = h-(2\,t_f)-(2\,R_1)$ =	280.58			
	d/t_w =	18.73			
for web	**Web is**	**Plastic**			
	Cross-section is	***Plastic***			

2. Check for resistance of cross-section to the combined effects fro yielding

Clause(9.3.1.2)

Factored applied axial force	N =	617	kN	*IS:800-2007*
Design strength	N_d =	7142.75	kN	*clause 9.3.1.1*
Design strength in Bending	M_{dy} =	528.67	kN m	*Page No-70*
	M_{dz} =	1084.57	kN m	
	$n = (N/N_d)$ =	0.09		
Design reduced flexural strength	M_{ndy} =	528.67	kN m	
	M_{ndz} =	1084.57	<= Md_z	

IS:800-2007 clause 9.3.1.1,Table-17, page no-71

Constants α_1 = 1

α_2 = 2

for plastic and compact section....**1**

$$\left(\frac{M_y}{M_{ndy}}\right)^{\alpha_1} + \left(\frac{M_z}{M_{ndz}}\right)^{\alpha_2} \leq 1.0$$

Interaction equation = 0.24

<= 1

Section is O.K.

Section is plastic so equation 1 is consider for design.

For semi compact section........**2**

$$\frac{N}{N_d} + \frac{M_y}{M_{ndy}} + \frac{M_z}{M_{ndz}} \leq 1.0$$

Interaction equation = 0.6

<= 1

Section is O.K.

3. Buckling resistance in compression

clause(7.1.2)

KL_z/r_{zz} = 37.85

KL_y/r_{yy} = 63.16

$\lambda_1 = \pi\,(E/f_y)^{0.5}$ = 75.67

Non dimensional effective slenderness ratio *IS:800-2007*

	$\lambda_z = (\lambda/\lambda_1)$ =	0.50		*clause 7.1.2.1*
	$\lambda_y = (\lambda/\lambda_1)$ =	0.83		*Page No-34*
	h/b_f =	0.99		
	t_f =	23.87	mm	
Imperfaction factor	α_z =	0.34	b	
	α_y =	0.49	c	
$\phi = 0.5\left[1+\alpha(\lambda-0.2)+\lambda^2\right]$	ϕ_z =	0.68		
	ϕ_y =	1.00		

Stress reduction factor

$$\chi = \frac{1}{\phi + (\phi^2 - \bar{\lambda}^2)^{\frac{1}{2}}} \leq 1.0$$

χ_z = 0.88 ≤ 1

χ_y = 0.64 ≤ 1

Design Compressive strength *IS:800-2007*

$P_d = (A_e \times f_y)/\gamma_{mo}$ *clause 7.1.2*

P_{dz} = 6314.93 kN *Page No-34*

P_{dy} = 4574.42 kN

Section is O.K.

4. Shear resistance of cross section

clause(8.4.1)

Design plastic shear resistance	$V_p = A_v(f_y/(3)^{0.5})/\gamma_{mo}$		
Load parallel to web due to moment @ **major axis**			
Design plastic shear resistance	$V_p =$ 999.03		
Maximum shear force on section	$V =$ 87.83		
Load parallel to flanges due to moment @ **minor axis**			
Design plastic shear resistance	$V_p =$ 3231.38		
Maximum shear force on section	$V =$ 0.00		*IS:800-2007*
			clause 8.4
	Section is O.K.		*Page No-59*
Shear buckling clause(8.4.2)			
	$d/t_w =$ 21.40	$> 67\varepsilon$	*IS:800-2007*
IS:800-2007 clause 8.4.2, page no-59			*clause 8.4.2*
	Section is O.K.		*Page No-59*

5. Buckling resistance in bending

(clause 8.2.2)

IS:800-2007 clause 8.2.2.1, page no-54

Elastic lateral buckling moment

$$M_{cr} = \frac{\pi^2 EI_y h_f}{2L_{LT}^2}\left[1+\frac{1}{20}\left[\frac{L_{LT}/r_y}{h_f/t_f}\right]^2\right]^{0.5}$$

$M_{cr} =$ 2822503294 N mm

$$f_{crb} = \frac{1.1\pi^2 E}{(L_{LT}/r_y)^2}\left[1+\frac{1}{20}\left[\frac{L_{LT}/r_y}{h_f/t_f}\right]^2\right]^{0.5}$$

$f_{crb} =$ 850.89

Non dimensional slenderness ratio

$$\lambda_{LT} = \sqrt{Z_{ez}\cdot f_y / M_{cr}}$$

$\lambda_{LT} = 0.64$

Imperfection parameter

$\alpha_{LT} =$ 0.21 *Page No-54*

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2\right]$$

$\phi_{LT} =$ 0.75

Bending stress reduction factor

$$\chi_{LT} = \left[\frac{1}{\phi_{LT} + \left\{\phi_{LT}^2 - \lambda_{LT}^2\right\}^{0.5}}\right] \le 1.0$$

$X_{LT} =$ 0.88 ≤ 1

Design bending compressive stress

$$f_{bd} = \chi_{LT}\, f_y / \gamma_m$$

$f_{bd} =$ 274.59 N/mm^2

Lateral torsional buckling resistance

$$M_d = Z_e f_{bd}$$

$M_{dz} =$ 949.55 kN m

$M_{dy} =$ 528.67 kN m

Section is O.K.

6. Buckling resistance in bending & axial comp.
(clause 9.3.2.2)

	$\psi_z = M_2/M_1 =$	0.00	
	$\psi_y = M_2/M_1 =$	0.00	
Equivalent uniform moment factor			
$C_{mz} = 0.6 + 0.4\,\psi \geq 0.4$	$C_{mz} =$	0.60	≥ 0.4
$C_{my} = 0.6 + 0.4\,\psi \geq 0.4$	$C_{my} =$	0.60	≥ 0.4
Equivalent uniform moment factor for lateral torsional buckling			
$C_{mLT} = 0.6 + 0.4\,\psi \geq 0.4$	$C_{mLT} =$	0.60	≥ 0.4
n_y	$K_y =$	1.00	
$k_z = 1+(\lambda_z-0.2)n_z \leq 1+0.8\,n_z$	$K_z =$	1.00	
	$n_y =$	0.13	
	$n_z =$	0.10	
	$K_{LT} =$	0.98	

Check No : 1	$\dfrac{P}{P_{dy}} + k_y \dfrac{C_{my} M_y}{M_{dy}} + k_{LT} \dfrac{M_z}{M_{dz}} \leq 1.0$	**0.68**	<=	1
Check No : 2	$\dfrac{P}{P_{dz}} + 0.6 k_y \dfrac{C_{my} M_y}{M_{dy}} + k_z \dfrac{C_{mz} M_z}{M_{dz}} \leq 1.0$	**0.43**	<=	1

Section is O.K.

B.3 Design as per IS:800 1984

Force y-kN	Moment x-kNm	Moment y-kNm	Moment z-kNm
502	416	0	0

Length of the member	L	=	6	m
Yield strength of steel	f_y	=	345	N/mm^2
Size of column		=	W 14X120	OK....
Modulus of easticity	E	=	200000	N/mm^2

Design of column :

Factor of safty for design	γ	=	1	
Axial Force	P	=	502	kN
Length of column	l_{ex}	=	6	m
	l_{ey}	=	6	m

Section Property :

Wt	W	=	179	kg/m
Area	A	=	22774	mm^2
Depth of section	D	=	368.3	mm
Width of flange	b	=	373.8	mm
Thk. Of flange	t_f	=	23.87	mm
Thk. Of web	t_b	=	14.98	mm
M.I. @ X-axis	I_{xx}	=	5.74E+08	mm^4
M.I. @ Y-axis	I_{yy}	=	2.06E+08	mm^4
Section modulus	Z_{xx}	=	3113540	mm^3
	Z_{yy}	=	1106130	mm^3
redius of gyration	r_{xx}	=	158.5	mm
	r_{yy}	=	95	mm
	h_1	=	254	mm

Axial compression

	$\sigma_{ac,cal}$	=	P/A	
		=	502000 / 22774	
		=	22.05	N/mm^2
$\lambda_x = l_{ex} / r_{xx}$	λ_x	=	37.86	
$\lambda_y = l_{ey} / r_{yy}$	λ_y	=	63.16	
conidered	λ	=	63.16	
5.1, IS-800-1984, Page No # 39	σ_{ac}	=	146.628	N/mm^2

Bending about X- axis

C_{mx} = 0.85

$\sigma_{bcx,\,cal.} = M_{xx}\ \ Y / I_{xx}$ $\quad\sigma_{bcx,\,cal}$ = 133.37 N/mm^2

$f_{ccx} = \pi^2 E / (\lambda_x^2)$ $\quad f_{ccx}$ = 1377.11

Kl_{ey} / r_{yy} = 63.16
D / T = 15.43
T / t = 1.6
d_1 / t = 16.96

Y = 664.35

$$X = Y\sqrt{1 + \frac{1}{20}\left(\frac{lT}{r_y D}\right)^2}$$

X = 900.63

k_1 = 1
k_2 = 0

$f_{cb} = k_1\ (X + k_2\ Y)\quad c_2/c$ $\quad f_{bc}$ = 900.63 N/mm^2

$$\sigma_{bcx} = \frac{0.66\ f_{cb}\ f_y}{\left[(f_{cb})^n + (f_y{}^n)\right]^{1/n}}$$

σ_{bcx} = 192.95 N/mm^2

Bending about Y- axis

C_{my} = 0.85

$\sigma_{bcy,\,cal.} = M_{yy}\ \ Y / I_{yy}$ $\quad\sigma_{bcy,\,cal}$ = 0 N/mm2

$f_{ccx} = \pi^2 E / (\lambda\ x^2)$ $\quad f_{ccx}$ = 494.82 N/mm^2

l_{ey} / r_{yy} = 63.16
D / T = 15.43
T / t = 1.6
d_1 / t 16.96

Y = 664.3403

$$X = Y\sqrt{1 + \frac{1}{20}\left(\frac{lT}{r_y D}\right)^2}$$

X = 900.61

k_1 = 1
k_2 = 0.5

$f_{bc} = k_1\ (X + k_2\ Y)\quad c_2/c_1$ $\quad f_{bc}$ = 1232.79 N/mm^2

$$\sigma_{bcy} = \frac{0.66\ f_{cb}\ f_y}{\left[(f_{cb})^n + (f_y{}^n)\right]^{1/n}}$$

σ_{bcy} = 203.78 N/mm^2

$$\frac{\sigma_{ac,cal}}{\sigma_{ac}} + \frac{C_{mx}\sigma_{bcx,cal}}{\left(1 - \frac{\sigma_{ac,cal}}{0.6 f_{ccx}}\right)\sigma_{bcx}} + \frac{C_{my}\sigma_{bcx,cal}}{\left(1 - \frac{\sigma_{ac,cal}}{0.6 f_{ccx}}\right)\sigma_{bcy}} \leq 1$$

0.10 + 0.60 + 0 = **0.70**
<= 1 **O.K....**

Appendix C

Varification example

Staad Pro V8i Input File[8] This example is varified with paper of "A Comparison of Frame Stability Analysis Methods in ANSI/AISC 360-05" by CHARLES J. CARTER and LOUIS F. GESCHWINDNER. Here the input file of STAAD ProV8i (20.07.07.19) ONE-BAY FRAME is presented.

STAAD PLANE

INPUT WIDTH 79

SET DISPLACEMENT 0.000235

UNIT FEET KIP

JOINT COORDINATES

1 0 0 0; 2 20 0 0; 3 0 15 0; 4 20 15 0;

MEMBER INCIDENCES

1 3 4; 2 1 3; 3 2 4;

MEMBER TRUSS

1

UNIT INCHES KIP

DEFINE MATERIAL START

ISOTROPIC STEEL

E 29000

POISSON 0.3

```
DENSITY 0.000283
ALPHA 6.5e-006
DAMP 0.03
END DEFINE MATERIAL
MEMBER PROPERTY AMERICAN
2 TABLE ST W14X90
3 TABLE ST W8X18
1 TABLE ST W8X18
CONSTANTS
MATERIAL STEEL ALL
SUPPORTS
1 FIXED
2 PINNED
DEFINE DIRECT ANALYSIS
FLEX 1 LIST 2
FYLD 50 LIST 2
AXIAL LIST 2
NOTIONAL LOAD FACTOR 0.002
END
UNIT FEET KIP
LOAD 1 LOADTYPE Dead TITLE LOAD CASE 1
JOINT LOAD
3 4 FY -200
3 FX 20
LOAD COMB 2 1.2D + 0.5L + 1.6W
1 1.0
LOAD 2 PDELTA
REPEAT LOAD
```

```
1 1.0
PERFORM ANALYSIS PRINT STATICS CHECK
PDELTA ANALYSIS CONVERGE
PERFORM DIRECT ANALYSIS LRFD ITERDIRECT 10 TAUTOL 0.01 DISPTOL
0.00278 -
REDUCEDEI 1 PDiter 15
PRINT JOINT DISPLACEMENTS ALL
PRINT MEMBER FORCES ALL
PRINT SUPPORT REACTION
LOAD LIST 2
PARAMETER
CODE AISC UNIFIED
METHOD LRFD
FYLD 7200 ALL
KY 1.0 MEMB 2
KZ 1.0 MEMB 2
CB 0.0 MEMB 2
TRACK 1 MEMB 2
CHECK CODE MEMB 2
FINISH
```

Compare the results with author's results and it is seen that all results match within 5% variation as shown in Table C.1.

Table C.1: Comparison of interaction ratio

Analysis Type	**Interaction ratio** of Paper	**Interaction ratio** From STAAD Pro.	**% Change**
1 st Order Analysis	0.84	0.88	+4.7
P-Delta Analysis	0.811	0.80	-1.3
Direct Analysis	0.796	0.758	+5.0

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