STABILITY ANALYSIS OF PIPE RACKS FOR INDUSTRIAL FACILITIES

By

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Stability Analysis of Pipe Racks for Industrial Facilities
Thesis directed by Professor Fredrick Rutz

ABSTRACT
Pipe rack structures are used extensively throughout industrial facilities worldwide. While stability analysis is required in pipe rack design per the AISC Specification for Structural Steel Buildings (AISC 360-10), the most compelling reason for uniform application of stability analysis is more fundamental. Improper application of stability analysis methods could lead to unconservative results and potential instability in the structure jeopardizing the safety of not only the pipe rack structure but the entire industrial facility.

The direct analysis method, effective length method and first order method are methods of stability analysis that are specified by AISC 360-10. Pipe rack structures typically require moment frames in the transverse direction creating intrinsic susceptibility to second order effects. This tendency for large second order effects demands careful attention in stability analysis. Proper application as well as clear understanding of the limitations of each method is crucial for accurate pipe rack design.
A comparison of the three AISC 360-10 methods of stability analysis was completed for a representative pipe rack structure using the 3D structural analysis program STAAD.Pro V8i. For the model chosen, all three methods of stability analysis met AISC 360-10 requirements.

For typical pipe rack structures, all three methods of stability analysis are acceptable as long as limitations are met and the methods are applied correctly. The first order method typically provided conservative results while the effective length method was determined to underestimate the moment demand in beams or connections that resist column rotation. The direct analysis method was found to be a powerful analysis tool as it requires no additional calculations to calculate additional notional loads, calculate effective length factors or verify AISC 360-10 limitations.

This abstract accurately represents the content of the candidate’s thesis. I recommend its publication.

Fredrick Rutz
ACKNOWLEDGEMENT

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1. **Introduction**

1.1 **Stability Analysis of Steel Structures**

The engineering knowledge base continues to grow and expand. This growth creates on-going challenges as designs demand adaptation in response to new information and technology. Although the value of stability analysis has long been recognized, implementation in design has historically been difficult as calculations were performed primarily by hand. Various methods were created to simplify the analysis and allow the engineer to partially include the effects of stability via hand calculations. However, with the development of powerful analysis software, rigorous methods to account for stability effects were developed. While stability analysis calculations can still be done by hand, most engineers now have access to software that will complete a rigorous stability analysis. The majority of the methods presented here assume that software analysis is utilized.

Stability analysis is a broad term that covers many aspects of the design process. According to the 2010 AISC Specification for Structural Steel Buildings (AISC 360-10) stability analysis shall consider the influence of second order effects (P-Δ and P-δ effects), flexural, shear and axial deformations, geometric imperfections, and member stiffness reduction due to residual stresses.
Both the 2005 and 2010 AISC Specification for Structural Steel Buildings recognize at least three methods for stability analysis: (AISC 360-05 and AISC 360-10)

1. First-Order Analysis Method
2. Effective Length Method
3. Direct Analysis Method

Other methods for analysis may be used as long as all elements addressed in the prescribed methods are considered.

Stability analysis is required for all steel structures according to AISC 360-10. The application of methods for stability analysis in design of structures varies greatly from firm to firm and from engineer to engineer. A crucial principle for engineers in the process of design is the inclusion of stability analysis in design. If stability analysis is not performed or a method of analysis is incorrectly applied, the ability of the structure to support the required load is potentially jeopardized. The analysis of nearly all complex structures is completed using advanced analysis software capable of performing various methods of analysis. Therefore omitting stability analysis in the design of structures creates unnecessary risk and is unjustified.
1.2 **Pipe Racks in Industrial Facilities**

Pipe racks are structures used in various types of plants to support pipes and cable trays. Although pipe racks are considered non-building structures, they should still be designed with the effects of stability analysis considered.

Pipe racks are typically long, narrow structures that carry pipe in the longitudinal direction. Figure 1-1 shows a typical pipe rack used in an industrial facility. Pipe routing, maintenance access, and access corridors typically require that the transverse frames are moment-resisting frames. The moment frames resist gravity loads as well as lateral loads from either pipe loads or wind and seismic loads. The transverse frames are typically connected using longitudinal struts with one bay typically braced. Any longitudinal loads are transferred to the longitudinal struts and carried to the braced bay. (Drake and Walter, 2010)
Pipe racks are essential for the operation of industrial facilities but because pipe racks are considered non-building structures, code referenced documents will usually not cover the design and analysis of the structure. The lack of industry standards for pipe rack design leads to each individual firm or organization adopting its own standards, many without clear understanding of the concepts and design of pipe rack structures. (Bendapodi, 2010) Process Industry Practices Structural Design Criteria (PIP STC01015) has tried to develop a uniform standard for design but it should be noted that this is not considered a code document.
The lack of code referenced documents can lead to confusion in the design of pipe racks. The concept of stability analysis should not be ignored based on the lack of code referenced documents AISC 360-10 should still be used as reference for stability analysis and design.
2. **Problem Statement**

2.1 **Introduction**

Industrial facilities typically have pipes and utilities running throughout the plant which require large and lengthy pipe racks. Pipe racks not only are used for carrying pipes and cable trays, but many times defines access corridors or roadways. It is relatively easy to add a braced bay in the longitudinal direction of a pipe rack because pipes and utilities run parallel to access roads. It is much more difficult to add bracing to the pipe rack in the transverse direction because of the potential for interference with pipes, utilities, corridors and access roads. Therefore moment connections in the transverse direction of the pipe rack are typically used. Figure 2-1 shows an elevation view of a length of pipe rack. Figure 2-2 shows a section view of the same pipe rack showing the moment resisting frame.

Pipe racks are a good example of structures that can be subject to large second order effects. The current AISC 360-10 defines three methods for stability analysis:

1. First Order Analysis Method
2. Effective Length Method
3. Direct Analysis Method

Limitations restrict practical application for certain methods.
Figure 2-1 Typical Elevation View of Pipe Rack

Figure 2-2 Section View Showing Moment Resisting Frame

Section - Moment Frame
2.2 **Significance of Research**

If stability analysis is not performed or a method is incorrectly applied, this could jeopardize the ability of the structure to support the required loads.

Most of the current literature on pipe racks discusses the application of loads and has suggestions on design and layout of pipe racks, while little applicable information is available on comparing the three methods of stability analysis for pipe racks. Currently the design engineer must research each method of stability analysis and decide which method to apply for analysis. After the analysis is completed, the engineer must then verify that the pipe rack meets all the requirements of the applied analysis method. If the requirements of AISC 360-10 methods are not met for the structure, then the engineer must completely reanalyze the structure using a new method of stability analysis which will meet the requirements. Comparing the various types of stability analysis will not only show the engineer which method will provide the most accurate analysis based on method limitations, but will also show why stability analysis is crucial.

2.3 **Research Objective**

The main purpose of this thesis will be to analyze various types of pipe rack structures, compare the results from stability analyses, and describe both positive and negative aspects of each method of stability analysis as it applies specifically to pipe
rack structures. The paper will also look at some of the various issues with applying each of the methods.

Some engineers are accustomed to braced frames structures, which are not susceptible to large second order effects, therefore those designers can tend to neglect or incorrectly apply methods of stability analysis. This thesis will not only show the importance of stability analysis, but also provide suggestions on practical implementation of each method. This could potentially save time in analysis and design because the process of selecting the appropriate stability analysis method will no longer be based on trial and error but rather on educated considerations that can easily be verified after analysis.
3. **Literature Review**

3.1 **Introduction**

This section will focus on review of the available literature on the subject of both pipe rack loading as well as stability analysis. Literature on the general theory of stability analysis will be reviewed. The main focus of this literature review will be on the three methods prescribed by AISC 360-10. Layout and loading guidelines for pipe racks will also be reviewed as this has a major influence on stability.

3.2 **Pipe Rack Loading**

3.2.1 **Load Definitions**

Pipe racks are unique structures that have unique loading when compared to typical buildings and structure. Pipe racks design is not covered under Minimum Design Loads for Buildings and Other Structures (ASCE 7-05) or International Building Code (IBC 2009) however the design philosophies should remain the same as that for all structures. Most company design criteria and Process Industry Practices (PIP) documents will list ASCE 7-05 or IBC as the basis for load definition and load combinations. There are several primary loads which should be considered in the design of pipe racks in addition to loads defined by ASCE 7-05 or IBC 2009. ASCE 7-05 primary load cases are as follows:
\( A_k = \) load or load effect arising from extraordinary event A

\( D = \) dead load

\( D_i = \) weight of ice

\( E = \) earthquake load

\( F = \) load due to fluids with well defined pressures and maximum heights

\( F_a = \) flood load

\( H = \) load due to lateral earth pressure, ground water pressure, or pressure of bulk materials

\( L = \) live load

\( L_r = \) roof live load

\( R = \) rain load

\( S = \) snow load

\( T = \) self-straining force

\( W = \) wind load

\( W_i = \) wind-on-ice determined in accordance with ASCE 7-05 Chapter 10

According to AISC 360-10, regardless of the method of analysis, consideration of notional loads is required. The notional loads may be required in all load combinations if certain requirements of the stability analysis are not satisfied.
The magnitude of notional load will vary based on the method used. Therefore the additional primary load cases per AISC 360-10 are as follows:

\[ N = \text{notional load per AISC, applied in the direction that provides the greatest destabilizing effect} \]

PIP STC01015 states that pipe racks shall be designed to resist the minimum loads defined in ASCE 7-05 as well as the additional loads described therein. PIP STC01015 breaks down the dead load into various categories that are not defined in ASCE 7-05. In addition, various loads from plant operation are defined and required for consideration in design.

PIP STC01015 breaks down the ASCE 7-05 Dead Load (D) by dividing the dead load into the subcategories listed below.

\[ D_s = \text{Structure dead load is the weight of materials forming the structure (not the empty weight of process equipment, vessels, tanks, piping nor cable trays), foundation, soil above the foundation resisting uplift, and all permanently attached appurtenances (e.g., lighting, instrumentation, HVAC, sprinkler and deluge systems, fireproofing, and insulation, etc…).} \]

\[ D_f = \text{Erection dead load is the fabricated weight of process equipment or vessels.} \]
**De** = Empty dead load is the empty weight of process equipment, vessels, tanks, piping, and cable trays.

**D_o** = Operating dead load is the empty weight of process equipment, vessels, tanks, piping and cable trays plus the maximum weight of contents (fluid load) during normal operation.

**D_t** = Test dead load is the empty weight of process equipment, vessels, tanks, and/or piping plus the weight of the test medium contained in the system.

PIP STC01015 also provides additional primary load cases from the effects of thermal loads caused from operational temperatures in the pipes.

**T** = Self-straining thermal forces caused by restrained expansion of horizontal vessels, heat exchangers, and structural members in pipe racks or in structures. This is essentially the same load case as defined in ASCE 7-05.

**A_f** = Pipe anchor and guide forces.

**F_r** = Pipe rack friction forces cause by the sliding of pipes or friction forces cause by the sliding of horizontal vessels or heat exchanges on their supports, in response to thermal expansion.
Seismic loads are also discussed in PIP STC01015. Seismic events can occur either when the plant is in operation or during shutdown when the pipes are empty. Therefore two seismic load cases are defined as follows:

\[ E_o = \text{Earthquake load considering the unfactored operating dead load and the applicable portion of the unfactored structure dead load.} \]

\[ E_e = \text{Earthquake load considering the unfactored empty dead load and the applicable portion of the unfactored structure dead load.} \]

### 3.2.2 Dead Loads

Further information on the dead loads specifically for pipe racks is defined in PIP STC01015. The operating dead load for piping on a pipe rack shall be 40 psf uniformly distributed over each pipe level. The 40 psf load is equivalent to 8 – inch diameter, schedule 40 pipes, full of water, at 15 inch spacing. For pipes larger than 8 inch, the actual load of pipe and contents shall be calculated and applied as a concentrated load.

The empty dead load \( (D_e) \) is defined for checking uplift and minimum load conditions. Empty dead load \( (D_e) \) is approximately 60% of the operating dead load \( (D_o) \) which is equivalent to 24 psf uniformly distributed over each pipe rack level. This is an acceptable approximation unless calculations indicate a different percentage should be used. (PIP STC01015)
Pipe racks for industrial applications are usually designed with consideration for potential future expansion. Therefore, additional space or an additional level should be provided and the rack should be uniformly loaded across the entire width to account for pipes that may be placed there in the future.

Cable trays are often supported on pipe racks and typically occupy a level within the rack specifically designated for cable tray. The operating dead load \( (D_o) \) for cable tray levels on pipe racks shall be 20 psf for a single level of cable tray and 40 psf for a double level. These uniform loads are based on estimates of full cable tray over the area of load application. (PIP STC01015)

The degree of usage for cable trays can vary greatly. The empty dead load \( (D_e) \) should be considered on a case by case basis. Engineering judgment should be used in defining the cable tray loading, because empty dead load \( (D_e) \) is defined for checking uplift and minimum load conditions.

3.2.3 **Live Loads**

Live load should be applied to pipe racks as needed. Pipe racks typically have very few platform or catwalks. When platforms are required for access to valves or equipment located on the pipe rack structure, the platform and supporting structure should be designed in accordance with ASCE 7-05 Live Loads.
3.2.4 **Thermal and Self Straining Loads**

Temperature effects on structural steel members should be included in design. PIP STC01015 introduces two additional self straining loads. These additional loads are caused by the operation effects on the pipes. The operational temperatures of pipes need to be considered in design.

Support conditions of pipes vary greatly and need to be considered in design. A pipe may be supported to resist gravity only, or may have varying degrees of restraint from guided in a single direction to fully anchored supports. Pipe stress analysis can be completed for all the pipes located in the pipe rack. This stress analysis takes into account the support type and location for each support and provides individual design forces for each pipe at that specific location. These resultant pipe loads can be used for design. However, application of loads in this manner does not include additional loads for futures expansion. Therefore, a uniform load at each level of the rack is typically applied in lieu of actual pipe forces. Local support condition should also be verified where large anchor forces are present.

3.2.5 **Snow Load and Rain Loads**

Snow loading should be considered in the design of pipe racks. Pipe racks typically do not have roofs or solid surfaces that large amounts of snow can collect on, therefore the engineer may reduce the snow load by a percentage using engineering judgment based on percentage of solid area and operational temperatures.
of pipes. Based on the reduced area for snow to accumulate, snow load combinations will usually not control the design of pipe racks. (Drake, Walter, 2010)

Rain loads are intended for roofs where rain can accumulate. Because pipe racks typically have no solid surfaces where rain can collect, rain load usually does not need to be considered in design of pipe racks. (Drake, Walter, 2010)

3.2.6 Wind Loads

ASCE 7-05 provides very little, if any guidance for application of wind load for pipe racks. The most appropriate application would be to assume the pipe rack is an open structure and design the structure assuming a design philosophy similar to that of a trussed tower. See Table 3-1 below for $C_f$, force coefficient. This method requires the engineer to calculate the ratio of solid area to gross area of one tower face for the segment under consideration. This may become very tedious for pipe rack structures because each face can have varying ratios of solids to gross areas.
Table 3-1 Force coefficient, $C_f$ for open structures trussed towers (Adapted from ASCE 7-05)

<table>
<thead>
<tr>
<th>Tower Cross Section</th>
<th>$C_f$</th>
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<tbody>
<tr>
<td>Square</td>
<td>$4.0\varepsilon^2 - 5.9\varepsilon + 4.0$</td>
</tr>
<tr>
<td>Triangle</td>
<td>$3.4\varepsilon^2 - 4.7\varepsilon + 3.4$</td>
</tr>
</tbody>
</table>

Notes:

1. For all wind directions considered, the area $A_f$ consistent with the specified force coefficients shall be the solid area of a tower face projected on the plane of that face for the tower segment under consideration.

2. The specified force coefficients are for towers with structural angles or similar flat sided members.

3. For towers containing rounded member, it is acceptable to multiply the specified force coefficients by the following factor when determining wind forces on such members: $0.51\varepsilon^2 + 5.7$, but not $> 1.0$

4. Wind forces shall be applied in the directions resulting in maximum member forces and reactions. For towers with square cross-sections, wind forces shall be multiplied by the following factor when the wind is directed along a tower diagonal: $1 + 0.75\varepsilon$, but not $> 1.2$

5. Wind forces on tower appurtenances such as ladders, conduits, lights, elevators, etc., shall be calculated using appropriate force coefficients for these elements.

6. Loads due to ice accretion as described in Section 11 shall be accounted for.

7. Notation:

$\varepsilon$: ratio of solid area to gross area of one tower face for the segment under consideration.

The method generally used for pipe rack wind load application comes from Wind Loads for Petrochemical and Other Industrial Facilities (ASCE, 2011). This report provides an approach for wind loading based on current practices, internal company standards, published documents and the work of related organizations.
Design wind force is defined as: (ASCE 7-05 Eqn 5.1)

\[ F = q_z \cdot G \cdot C_f \cdot A \]

With:

- \( q_z \) = Velocity pressure determined from ASCE 7-05 Section 6.5.10
- \( G \) = Gust effect factor determined from ASCE 7-05 Section 6.5.8
- \( C_f \) is defined as the force coefficient and varies based on the shape and direction of wind. Structural members can have force coefficients between 1.5 and 2. \( C_f \) can be taken as 1.8 for all structural members or equal to 2 at and below the first level and 1.6 above the first level. No shielding shall be considered. \( C_f \) for pipes should be 0.7 as a minimum. \( C_f \) for cable should be taken as 2.0. (ASCE, 2011)

These values of \( C_f \) are developed based on the Table 3-2 below. Cable tray are considered square in shape with \( h/D = 25 \) corresponding to \( C_f = 2.0 \). Pipe are round in shape with \( h/D = 25 \) and a moderately smooth surface corresponding to \( C_f = 0.7 \).
Table 3-2 $C_f$ force coefficient (Adapted from ASCE 7-05)

<table>
<thead>
<tr>
<th>Cross-Section</th>
<th>Type of Surface</th>
<th>h/D</th>
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<td></td>
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<td>1</td>
</tr>
<tr>
<td>Square (wind normal to face)</td>
<td>All</td>
<td>1.3</td>
</tr>
<tr>
<td>Square (wind along diagonal)</td>
<td>All</td>
<td>1</td>
</tr>
<tr>
<td>Hexagonal or octagonal</td>
<td>All</td>
<td>1</td>
</tr>
<tr>
<td>Round ($D/\sqrt{q_z} &gt; 2.5$)</td>
<td>Moderately smooth</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>Rough ($D'/D = 0.02$)</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td>Very rough ($D'/D = 0.08$)</td>
<td>0.8</td>
</tr>
<tr>
<td>Round ($D/\sqrt{q_z} \leq 2.5$)</td>
<td>All</td>
<td>0.7</td>
</tr>
</tbody>
</table>

Notes:

1. The design wind force shall be calculated based on the area of the structure projected on a plane normal to the wind direction. The force shall be assumed to act parallel to the wind direction.

2. Linear interpolation is permitted for h/D values other than shown.

3. Notation:
D: Diameter of circular cross-section and least horizontal dimension of square, hexagonal or octagonal cross-section at elevation under consideration in feet
D’: Depth of protruding elements such as ribs and spoilers, in feet
h: Height of structure, in feet
$q_z$: Velocity pressure evaluated at height $z$ above ground, in pounds per square foot

The tributary area (A) for pipes is based on the diameter of the largest pipe (D) plus 10% of the width of the pipe rack (W), then multiplied by the length of the pipes (L) (usually the spacing of the bent frames). The tributary area for pipes is the projected area of the pipes based on wind in the direction perpendicular to the length of pipe. Wind load parallel to pipe is typically not considered in design since there is typically very little projected area of pipe for applying wind pressure. (ASCE, 2011)
\[ A = L(D+0.1W) \]

The tributary area takes into account the effects of shielding on the leeward pipes or cable tray. The 10% of width of pipe rack is added to account for the drag of pipe or cable tray behind the first windward pipe. It is based on the assumption that wind will strike at an angle horizontal with a slope of 1 to 10 and that the largest pipe is on the windward side. (ASCE, 2011)

The tributary area for structural steel members and other attachments should be based on the projected area of the object perpendicular to the direction of the wind. Because the structural members are typically spaced at greater distances than pipes, no shielding effects should be considered on structural members and the full wind pressures should be applied to each structural member.

The gust effect factor \( G \), and the velocity pressure \( q_v \), should be determined based on ASCE 7-05 sections referenced above.

3.2.7 **Seismic Loads**

Pipe racks are typically considered non-building structures, therefore seismic design should be carried out in accordance with ASCE 7-05, Chapter 15. A few slight variations from ASCE 7-05 are recommended. The operating earthquake load \( E_o \) is developed based on the operating dead load as part of the effective seismic weight. The empty earthquake load \( E_e \) is developed based on the empty dead load as part of the effective seismic weight. (Drake and Walter, 2010)
The operating earthquake load and the empty earthquake load are discussed in more detail in the load combinations for pipe racks. Primary loads, $E_o$ and $E_e$ are developed and used in separate load combinations to envelope the seismic design of the pipe rack.

ASCE Guidelines for Seismic Evaluation and Design of Petrochemical Facilities (1997) also provides further guidance and information on seismic design of pipe racks. The ASCE guideline is however based on the 1994 Uniform Building Code (UBC) which has been superseded in most states by ASCE 7-05 or ASCE 7-10. Therefore the ASCE guideline should be considered as a reference document and not a design guideline.

3.2.8 Load Combinations

Based on the inclusion of additional primary load cases as specified by PIP STC01015, additional load combinations need to be considered. PIP STC01015 specifies load combinations to be used for pipe rack design. Both LRFD and ASD load combinations are specified. LRDF load combinations will be the focus of this section as AISC LRFD will be used for analysis and design. ASD load combinations should be considered when checking serviceability limits on pipe racks.

Because additional primary load cases are included in the design and ASCE 7-05 does not govern the design of pipe racks because they are typically considered non-building structures, PIP STC01015 load combinations should be used. In
practice, PIP STC01015 load combinations and ASCE 7-050 load combinations are very similar and a combination of the specified load combinations can be used. ASCE 7-05 primary load cases must be redefined with the additional subcategories of loads defined by the general primary load cases. Example: Dead load as defined by ASCE 7-05 needs to be broken down into additional primary load cases such as the dead load of the structure, the dead load of the empty pipe, etc…

PIP STC01015 LRFD load combinations specified for pipe racks are listed below:

1. \(1.4(D_s+D_o+F_t+T+A_f)\)
2. \(1.2(D_s+D_o+A_f)+(1.6W \text{ or } 1.0E_o)\)
3. \(0.9(D_s+D_o)+1.6W\)
4. a) \(0.9(D_s+D_o)+1.2A_f+1.0E_o\)
   b) \(0.9(D_s+D_o)+1.0E_e\)
5. \(1.4(D_s+D_t)\)
6. \(1.2(D_s+D_t)+1.6W_p\)

ASCE 7-05 LRFD load combinations are listed below:

1. \(1.4(D+F)\)
2. \(1.2(D+F+T)+1.6(L+H)+0.5(L_r \text{ or } S \text{ or } R)\)
3. \(1.2D+1.6(L_r \text{ or } S \text{ or } R)+(L \text{ or } 0.8W)\)
4. \(1.2D+1.6W+L+0.5(L_r \text{ or } S \text{ or } R)\)
5. \(1.2D+1.0E+L+0.2S\)
6. 0.9D+1.6W+1.6H 
7. 0.9D+1.0E+1.6H 

When comparing the two sets of load combinations, there are some similarities. Certain primary loads such as live load, live roof load, snow load and rain load do not typically apply or control the design of the pipe racks, therefore most load combinations with these primary load cases will not control the design. Therefore ASCE 7-05 load combination 2 and 3 will not be considered in design. 

Taking into account the subcategories of primary load cases used in PIP STC01015, ASCE 7-05 load combinations can be compared directly and a comprehensive list of all load combinations can be developed.

Below is listed the combined load combinations to be used in this research for design of pipe racks referenced from PIP STC01015. Reference of specific load combination number from ASCE 7-05 is also included if applicable.

1. 1.4(D_s+D_o+F_r+T+A_f) - ASCE 1
2. 1.2(D_s+D_o+A_f)+(1.6W or 1.0E_o) - ASCE 4 and 5
3. 0.9(D_s+D_o)+1.6W – ASCE 6
4. a) 0.9(D_s+D_o)+1.2A_f+1.0E_o – ASCE 7
   b) 0.9(D_s+D_o)+1.0E_e
5. 1.4(D_s+D_t)
6. 1.2(D_s+D_t)+1.6W_p
It can be seen in the above load combinations that ASCE 7-05 load combinations 1, 4, 5, 6 and 7 are covered by the PIP STC01015 load combinations. Slight changes such as the inclusion of $A_f$ are added to load combinations per the direction of PIP STC01015. Additional load combinations to cover test load conditions, partial wind, $W_p$, during test and seismic on the empty condition are covered by PIP. Engineering judgment should be used to determine if any additional load combinations should be considered in design.

ASD load combinations from both PIP STC01015 and ASCE7-05 are combined in a similar fashion to come up with a combined list of load combinations used for design.

3.3 **Column Failure and Euler Buckling**

An ideal column is considered to be perfectly straight with the load applied directly through the centroid of the cross section. Theoretically the load on an ideal column can increase until the limit state occurs by yielding or rupture. Figure 3-1 shows a graph of axial load “P” vs lateral deflection “y”. The axial load is increased until yielding occurs with no lateral deflection.
For slender columns, this yielding is never reached. The axial load is increased to a point of critical loading where the column is on the verge of becoming unstable. The critical load is determined as the point where, if a small lateral load (F) were applied at the mid-span of the column, the column would remain in the deflected position even after the lateral load was removed. Any additional load will cause further lateral displacement. This is shown in Figure 3-2.
This critical load for slender columns is based on Euler buckling. Euler buckling load is the theoretical maximum load that an ideal pin ended column can support without buckling. (Euler, 1744) It is stated as:

\[ P_{cr} = \frac{\pi^2 \cdot E \cdot I}{L^2} \]

- \( P_{cr} \) = Euler Buckling Load or Critical Buckling Load
- \( L \) = Length of Column
- \( E \) = Modulus of Elasticity
- \( I \) = Moment of Inertia of Column

Bifurcation is the point when the column is in a state of neutral equilibrium as the critical buckling load is applied to the column. At the point of bifurcation, the column is on the verge of buckling. Instead of the graph shown in Figure 3-1, the
graph now is shown in Figure 3-3. The load is increased to the critical load where the column becomes unstable and buckling can occur. (Hibbeler, 2005)

![Graph showing Load vs. Deflection – Euler Buckling](image)

**Figure 3-3 Load vs. Deflection – Euler Buckling**

Euler’s formula for critical load was derived based on the assumption of an ideal column. However, ideal columns do not exist. The load is never applied directly through the centroid and the column is never perfectly straight. The existence of load eccentricities, out of plumb members, member geometric imperfections, material flaws, residual stresses, therefore second order effects become the basis for stability analysis. Based on the discussion above, most real columns will never suddenly buckle but will slowly bend due to the eccentricities and out of straightness. (Hibbeler, 2005)
3.4 Stability Analysis

3.4.1 AISC Specification Requirements

AISC 360-10 Specification for Structural Steel Building states in section C1.

Stability shall be provided for the structure as a whole and for each of its elements. The effects of all of the following on the stability of the structure and its elements shall be considered: (1) flexural, shear and axial member deformations, and all other deformations that contribute to displacements of the structure; (2) second-order effects (both P-Δ and P-δ effects); (3) geometric imperfections; (4) stiffness reductions due to inelasticity; and (5) uncertainty in stiffness and strength. All load-dependant effects shall be calculated at a level of loading corresponding to LRFD load combinations of 1.6 times ASD load combinations.

3.4.2 Second Order Effects

Second order effects are a means to account for the increase in forces based on the deformed shape of the member or frame. Second order effects can be further broken down to P-δ and P-Δ effects. P-Δ effects are the effects of loads acting on the displaced location of joints or nodes in a structure. P-δ effects are the effects of loads acting on the deflected shape of a member between joints or nodes. Figure 3-4b shows the effects of P-δ and Figure 3-4a shows the effects of P-Δ. (AISC 360-10) (Ziemian, 2010)
Second order effects are typically the driving factor for stability analysis. All the other requirements for stability design are implemented based on the effect that they will have on the second order effects. The importance of second order effects becomes readily apparent when comparing an example frame using first order elastic analysis and a second order elastic analysis as shown in Figure 3-5. Lateral displacements and as a result, member forces, can be drastically underestimated if a first order analysis is performed.
Figure 3-5 Comparison of First Order Analysis to Second Order Analysis

(Adapted from Gerschwindner, 2009)

Second order analysis can be carried out per AISC 360-10 methods of either rigorous second order analysis or the approximate second order analysis, both of which are discussed in more detail in further sections.

The importance of second order effects on overall stability of structure can be seen in various structural failures over the years. For example, in 1907, the Quebec Bridge collapsed during construction due to failure of the compression chords of the
truss. In the weeks previous to the collapse, deflections in the chords was noticed and reported but nothing was done and work continued until the eventual collapse of the bridge. While many factors led to the failure of the bridge, one of the errors made in the design of the bridge was in the design of the compression chords. The compression chords were fabricated slightly curved for aesthetic reasons. However, because this member curvature complicated the calculations, the members were designed as straight members. Therefore, the actual second order effects were increased and the buckling capacity was reduced when compared to the straight member as designed. (Delatte, 2009) As can be seen from Figure 3-4 and 3-5, the second order effects can increase both the displacement and moments to failure much before Euler buckling load is ever reached. Figure 3-6 shows the Quebec Bridge prior to collapse and Figure 3-7 shows the result of the failure.

Figure 3-6 Quebec Bridge Prior to Collapse (Canada, 1919)
Some general points concerning second order analysis made by Ziemian (2010) are as follows:

1. Second order behavior can affect all components and internal member forces within a structure.

2. Second order moments do not necessarily have the same distribution as the first order moments and therefore the first order moments cannot be simply amplified. LeMessurier (1977) and Kanchanalai and Lu (1979) however give several practical applications where amplification of first order moments to achieve an approximation of second order moments is applicable.
3. All structures will experience both P-δ and P-Δ effects but the magnitude of the effects will vary greatly between structures.

4. Linear superposition of effects cannot be used with second order analysis; the response is non-linear.

3.4.3 **Flexural, Shear and Axial Deformation**

Second order effects are required in stability analysis as explained in the previous section. Accurate deformation must be calculated because second order effects are based on deformation to determine the amplified moments and forces.

The AISC 360-10 requires that flexural, shear and axial deformations be considered in the design. Although flexural deformations will usually be the largest contributor to overall structural deformation, axial and shear deformations should not be ignored. Figure 3-8 shows a frame and the calculated deformations from flexural, shear and axial. If shear and axial deformations were ignored in the design and analysis, the amplifications of moments and forces from second order effects could be underestimated and the structure could become unstable.
3.4.4 Geometric Imperfections

Geometric imperfections refer to the out-of-straightness, out-of-plumbness, material imperfections and fabrication imperfections. The maximum allowable geometric imperfections are set by AISC Code of Standard Practice for Steel Building and Bridges. The main geometric imperfections of concern in stability analysis are member out-of-straightness and frame out-of-plumbness. Member out-of-straightness is limited to L/1000, where L is the member length between brace or frame points.
Frame out-of-plumbness is limited to H/500, where H is the story height. (AISC 360-10) (AISC 303-10)

Geometric imperfections cause eccentricities for axial loads in the structure and members. These eccentricities need to be accounted for in stability analysis because they can cause destabilizing effects and increased moments. Eccentricities also increase the second order effects in the analysis of the structure. Maximum tolerances specified by AISC Code of Standard Practice for Steel Buildings and Bridges should be assumed for analysis unless actual imperfection values are known. (AISC 360-10)

Columns are never perfectly straight and contain either member out-of-straightness or general out-of-plumbness, therefore an initial eccentricity of the axial load will be experienced. Figure 3-9 shows how the real column will behave with an initial imperfection ($y_0$). Nominal column capacity ($P_n$) will be reached well before Euler Buckling load ($P_{cr}$). Second order effects are included in this graph of axial load vs. deformation. Based on the presence of the initial displacement, moment will develop and the column will typically yield based on flexure and compression while the theoretical Euler buckling will never be reached.
3.4.5 Residual Stresses and Reduction in Stiffness

Residual stresses are internal stresses contained in a structural steel member. There are several sources of residual stresses: (Salmon and Johnson, 2008)

1. Uneven cooling after hot rolling of the structural member.
2. Cold bending or cambering during fabrication.
3. Punching holes or cutting during fabrication.
4. Welding.

Uneven cooling and welding typically produce the largest residual stresses in a member. Local welding for connections does produce residual stresses but the presence of these stresses tend to be localized and are not considered in overall column or beam design strength. Residual stress from uneven cooling happens when the rolled shape is cooled at room temperature from the rolling temperatures. Certain
areas of the member will cool more rapidly than others. For example, the flange tips of a wide flange shape are surrounded by air on three sides and cool more rapidly than the material at the junction of the flange and web. As the flange tips cool, they can contract freely because the other regions have yet to develop axial stiffness. When the slower cooling sections begin to cool and contract, the axial stiffness from the cooled regions restrains the contraction thus creating compression on the faster cooling section and tension in the slower cooling sections. Figure 3-10 shows the residual stresses typically seen in hot rolled wide flange shapes from uneven cooling. (Vinnakota, 2006) (Huber and Beedle, 1954) (Yang et al., 1952)

![Residual Stress Patterns in Hot Rolled Wide Flange Shapes](image)

Welding of built up sections produces residual stresses as a result of the localized heating applied during the welding. A built up wide flange shape will have
compression on the flange tips and middle of the web and tension around the junction of the web and flange. (Vinnakota, 2006)

The presence of residual stresses results in a non-linear behavior of the stress strain curve. The average yield stress of the section is reduced by the amount of residual stress in the member. Therefore the section will start to yield before the stress reaches the theoretical yield stress of a member with no residual stress. Linear elastic behavior is experienced to the point of theoretical yield stress ($F_y$) minus the residual stress (see Figure 3-11). After this point, non-linear behavior is experienced and plasticity begins to spread through the section. (Salmon and Johnson, 2008)

Figure 3-11 Influence of Residual Stress on Average Stress-Strain Curve (Salmon and Johnson, 2008)
Residual stresses need to be considered in stability analysis because of the effect of general softening of the structure from the spread of plasticity through the cross section causing reduced stiffness. The reduced stiffness increases deflections and therefore increases the second order effects on the structure. (AISC 360-10)

Beam and column design strength is calculated based on empirical equations which take into account the residual stress which is assumed to follow a Lehigh pattern which is a linear variation across the flanges and uniform tension in the web. The AISC 360-10 strength equations were developed and calibrated based on research from Kanchanalai (1977) and ASCE Task Committee (1997). Figure 3-12 shows the idealized residual stresses for typical wide flange sections which follows the Lehigh pattern. The residual stresses are assumed to be $0.3F_y$ in wide flange shapes. (AISC 360-10) (Ziemian, 2010) (Deierlien and White, 1998)

![Figure 3-12 Idealized Residual Stresses for Wide Flange Shape Members – Lehigh Pattern (Adapted from Ziemian, 2010)](image-url)
3.5 AISC Methods of Stability Analysis

As discussed before, AISC 360-10 states that any method that considers the influence of second-order effects, flexural, shear and axial deformation, geometric imperfections, and member stiffness reduction due to residual stresses on the stability of the structure and its elements is permitted.

Various types of methods of have been developed and AISC 360-10 detailed the requirements for a few of these methods. Each method listed, does in some way, address all the various requirements specified by AISC 360-10. All methods listed in AISC 360-10, excluding the first order analysis method, require a second order analysis. Table 2-2 from AISC 360-10 provides a summary of requirements and limitations of each of the methods. AISC 360-10 allows two types of second order analysis; Approximate Second Order Analysis and Rigorous Second Order Analysis.

Both methods of second order analysis either accurately account for or approximate geometric nonlinear behavior. In reality, geometric nonlinear behavior is only one of the nonlinear types of behavior that should be considered in design. Material nonlinear behavior (inelastic analysis) caused by reduction in stiffness should also be considered in design. Figure 3-13 shows the results from various types of analyses.
While software is available that performs a true second order inelastic analysis, it is very computationally expensive, and therefore other measures must be considered in analysis and design. AISC 360-10 strength equations are typically based on the results from a second order elastic analysis. The AISC LRFD general approach for strength and stability can be represented by the following equation:

$$\Sigma \gamma_i Q_i \leq \Phi R_n$$
The left hand side of the formula represents the effects of factored loads on a structural member, connection and the right side represents the design resistance or design strength of the specified element with:

\[ Q_i = \text{internal forces created by applied load} \]

\[ R_n = \text{Nominal member or connection strength} \]

\[ \gamma_i = \text{factor to account for variability in load (load factor)} \]

\[ \Phi = \text{factor to account for variability in resistance (strength reduction factor)} \]

Geometric nonlinear behavior can be accounted for using a second order elastic analysis (left side of the equation). These load effects are then compared to resistance based on material and geometric inelasticity (right side of the equation). (Yura el al., 1996) As Figure 3-13 shows, a direct comparison of load effects from second order elastic analysis and member resistance is not compatible because the inelastic material deflections are not considered in an elastic second order analysis. Therefore AISC 360-10 design equations should be calibrated for the results of an elastic second order analysis or the effects of material inelasticity must be accounted for in the elastic second order analysis. (Ziemian, 2010) This can be accomplished through various methods discussed in more detail in further sections.

### 3.5.1 Rigorous Second Order Elastic Analysis

To fully capture the second order effects as described in previous sections, non-linear geometric behavior should be accurately calculated. With the constant
increase of computational capabilities, rigorous second order analysis is becoming much more common in design practice. It should be noted that the AISC 360-10 definition of rigorous second order analysis is typically not meant to represent a true non-linear second order analysis but will still produce results that accurately calculate the second order effects. Many methods can be used for analysis but the general form is usually expressed as: (Ziemian, 2010)

\[ \{dF\} - \{dR\} = K \{d\Delta\} \]

With:

\{dF\} = Vector of incremental applied nodal forces

\{dR\} = Vector of unbalanced nodal forces, difference between current internal forces and applied loads

K = Stiffness matrix

\{d\Delta\} = Vector of incremental nodal displacements and rotations

Most solutions use an iterative approach for solving for second order effects. The unbalanced forces are calculated based on the deformed geometry at the end of each iteration and used as the basis for the next iteration. Iterations can be performed until the unbalanced force vector is determined to be negligible. Figure 3-14 shows a method commonly referred to as the Newton-Raphson incremental – iterative solution.
Additional methods based on the Newton-Raphson incremental-iterative solution have been developed based on the limitation imposed by the use of this solution. McQuire et al. (2000) and Chen and Lui (1991) have provided general overview of additional methods.

A variation of these methods that can be used by STAAD.Pro V8i is referred to as the Lagrangian procedure. This procedure revises the stiffness matrix, $K$, to take the following form:

$$ [K] = [Ke] + [Kg] $$

\[ \text{Figure 3-14 Visual Representation of Incremental – Iterative Solution Procedure (Adapted from Ziemian, 2010)} \]
With Ke being the standard linear elastic stiffness matrix and Kg being the geometric stiffness matrix. This method recognizes that as the structure deforms the stiffness associated with the member forces is changed at each increment of the solutions. This leads to more accurate results and the ability to use this type of solution for dynamic analyses because the method accounts for change in the natural period due to second order effects and the stiffening of the structure. (Galambos, 1998) This procedure is given in more detail by Crisfield (1991), Yang and Quo (1994) and Bathe (1996). STAAD Technical Manual (2007) also gives details of how this method is applied for analysis.

3.5.2 **Approximate Second Order Elastic Analysis**

In lieu of a rigorous second order elastic analysis which is iterative and can demand extensive computational effort, approximate second order analysis methods have been developed over the years to potentially simplify analysis. Rutenberg (1981, 1982), White et al (2007a,b) and LeMessurier (1976, 1977) have each developed approximate methods of analysis to either simplify analysis in computer applications or simplify hand calculations. (Ziemian, 2010)

The AISC 360-10 method of approximate second order analysis uses amplification factors B1 and B2 to account for P-δ and P-Δ effects, respectively. See AISC 360-10 Appendix 8 for the procedure.
Because this approximate second order analysis is typically used for hand calculations of frame analysis, no additional discussion will ensue based on the assumption that the engineer has access to software capable of a rigorous second order analysis.

3.5.3 Direct Analysis Method

Introduced in AISC 360-05, the direct analysis method represents a fundamentally new alternative to traditional stability analysis methods. (Griffis and White, 2010) The most significant development addressed in this method is that column strength can be based on the unbraced length of the member therefore eliminating the need to calculate the effective length of the member (K may be taken as 1 for all members). (Ziemian, 2010)

While all the requirements for AISC stability analysis are covered with this method, slight variations in each requirement are allowed and discussed in further detail. One major advantage of the direct analysis method is that it has been developed and verified for application to all types of structural systems and therefore has no limitations for use. (Maleck and White, 2003)

Accurate second order analysis is the cornerstone of the direct analysis method. As previously discussed, two types of second order analysis are allowed by AISC 360-10, rigorous second order analysis and approximate second order analysis. As with all AISC 360-10 methods that use second order analysis results, the direct
analysis method is built around the assumption of LRFD loads and therefore if ASD loads are to be used in analysis, they must be multiplied by 1.6 before second order analysis is completed because of the nonlinearity of second order effects. (AISC 360-10) (Nair, 2009)

Flexural, shear and axial deformations need to be considered in analysis. As previously discuss, accurate deformations are needed for calculation of second order effects. However, in discussion of shear and axial deformation AISC 360-10 explicitly uses the word “consider” instead of “include”. This allows the engineer to ignore certain deformations based on the type of structural system. For example, the shear deformations could feasibly be neglected in a low rise moment frame and produce results with an error of less than 3%. High rise moment frame systems on the other hand, could produce much higher errors if shear deformations were to be neglected. (AISC 360-10) Most modern analysis software is capable of calculating accurate flexure, shear and axial deformations, and very little effort by the engineer is required to achieve the most accurate results available by the software.

The direct analysis method is calibrated on the assumption that geometric imperfections are equal to the maximum material, fabrication and erection tolerances permitted by the Code of Standard Practice for Steel Buildings and Bridges (AISC 303-10). Geometric imperfection may be accounted for by two methods; direct modeling of imperfections or notional loads. (AISC 360-10) Direct modeling of imperfections can become quite tedious because as a minimum, four models must be
developed, each with the deflection in one of the four principle directions with the additional member out of straightness modeled corresponding to the worst case direction. Notional loads are defined as horizontal forces added to the structure to account for the effects of geometric imperfections. (Ericksen, 2011) For the direct analysis method the magnitude of notional loads applied to the structure is 0.2% of the total factored gravity load at each story.

\[ N_i = 0.002\alpha Y_i \]

With:

\[ \alpha = \begin{cases} 1.0 \text{ (LRFD)}; & 1.6 \text{ (ASD)} \end{cases} \]

*\( N_i \) = Notional lateral load applied at level i, kips

*\( Y_i \) = Gravity load applied at level i from the LRFD or ASD load combinations as applicable, kips

It can be see that 0.2% of gravity loads is appropriately selected as 1/500 which also corresponds to the maximum out-of-plumbness for columns from AISC 303-10. Analysis will show that either applying a notional load of 0.2% or directly modeling out-of-plumbness will produce similar results. (Malek and White, 1998) Member out-of-plumbness is accounted for by notional loads, but member out-of-straightness still needs to be considered in design. AISC 360-10 has developed the column strength equations based on maximum out-of-straightness tolerances. (Ziemian, 2010) (White et al., 2006)
For the direct analysis method, notional loads should be applied in combination with all gravity and lateral load combinations to create the worst effects. AISC 360-10 does however, allow notional loads to be applied only to gravity load combinations as long as the ratio of second order to first order drifts does not exceed 1.5 using the unreduced elastic stiffness or 1.7 if the reduced elastic stiffness is used in analysis. The errors seen by this simplification are relatively small as long as the ratio of drifts remains below the specified limits. (AISC 360-10)

Reduction in stiffness of the structure is caused by partial yielding of members. This yielding is further accentuated by residual stresses. The direct analysis method specifies reduced stiffnesses of $EI^*$ and $EA^*$ with:

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

$$EA^* = 0.8EA$$

The reduced stiffness factor of 0.8 is applied for two reasons. The first and most readily apparent is the reduction in stiffness in intermediate and stocky members, namely columns, due to inelastic softening of members before they reach their design strength. The second reason relates to slender members that are governed by elastic stability. 0.8 is roughly equivalent to the product of $\Phi = 0.9$ and the factor 0.877. These factors are used in development of the AISC column curve (AISC 360-10 Eqn. E3-3) which is modified by the above factors for slender elements to account for member out-of-straightness. (Ziemian, 2010) (AISC 360-10)
The $\tau_b$ value is an adjustment factor to account for additional reduction in stiffness in cases where high axial stresses are present which can reduce the bending stiffness of the member. (AISC 360-10)

$$\tau_b = 1.0 \text{ when } \alpha P_r/P_y \leq 0.5$$

$$\tau_b = 4(\alpha P_r/P_y)[1-(\alpha P_r/P_y)] \text{ when } \alpha P_r/P_y \geq 0.5$$

where:

$$\alpha = 1.0 \text{ (LRFD); } 1.6 \text{ (ASD)}$$

$$P_r = \text{ Required axial compressive strength using LRFD or ASD load combinations.}$$

$$P_y = \text{ Axial Yield strength (}A_g*F_y)$$

AISC 360-10 does allow $\tau_b = 1.0$ for all cases if the notional load is increased by $0.1\%Y_i$. This additional notional load is meant to increase the lateral deformation to envelope the effects caused by reduction in stiffness in high axial loaded members. However, Powell notes that this method does not appear to be logical because notional loads are meant to account for initial out of plumbness and not for reduction in stiffness (Powell, 2010). Figure 3-15 shows a graphical representation of the effect of $\tau_b$ on the reduction in stiffness.
3.5.4 Effective Length Method

In recent years, the traditional method for stability analysis has been the effective length method. In general, the effective length method calculates the nominal column buckling resistance using an effective length (KL) and the load effects are calculated based on either a rigorous or approximate second order analysis. (Ziemian, 2010)

As with the direct analysis method, the effective length is built around determining accurate second order effects. This can be done by either a rigorous second order analysis or by an approximate second order analysis. (AISC 360-10) Both methods of second order analysis are based on the assumption that flexural, shear and axial deformations are considered in calculations of second order effects.
Prior to AISC 360-05, there were few limitations on applications of the effective length method. Several studies by Deierlein et al. (2002), Maleck and White (2003), and Surovek-Maleck and White (2004a and 2004b) have shown that use of the effective length method could produce significantly unconservative results in certain types of framing systems. Therefore AISC 360-05 imposed additional requirements and limitations.

1. Notional loads need to be included in gravity only load combinations to account for member out-of-plumbness.

2. The ratio of second order drift to first order drift or $B_2$ is limited to 1.5.

Geometric imperfections are covered by applications of notional loads or direct modeling of imperfections for analysis. Notional loads are applied in the same manner as described with the direct analysis method with: (AISC 360-10)

$$N_i = 0.002\alpha Y_i$$

However, based on the limitation of the ratio of second order drift to first order drift or $B_2$, by definition, notional loads need only be applied to gravity load combinations.

One of the main components of the effective length method is the calculation of the effective length factor $K$. The most common method for determining $K$ is through the use of the alignment charts found in the commentary for Appendix 7 in AISC 360-10 which can be seen in Figures 3-16 and 3-17.
Figure 3-16 Alignment Chart – Sidesway Inhibited (Braced Frame) (AISC 360-10)
The alignment charts were developed based on the following assumptions:

1. Behavior is purely elastic.
2. All members have constant cross section.
3. All joints are rigid.
4. For columns in frames with sidesway inhibited, rotations at opposite ends of the restraining beams are equal in magnitude and opposite in direction, producing single curvature bending.
5. For columns in frames with sidesway uninhibited, rotation at opposite ends of the restraining beams are equal in magnitude and direction, producing reverse curvature bending.

6. The stiffness of parameter $L \sqrt{(P/EI)}$ of all columns is equal.

7. Joint restraint is distributed to the column above and below the joint in proportion to $EI/L$ for the two columns.

8. All columns buckle simultaneously

9. No significant axial compression force exists in the girders.

The assumptions listed above, seldom if ever are seen in a real structure and therefore additional methods of determining $K$ have been developed. Geschwindner(2002) and ASCE Task Committee (1997) have provided an overview of the various methods for determining accurate values of $K$. In addition to methods listed in AISC 360-10, Yura (1971) and LeMessurier (1995) presented various approaches for calculation of the effect length factor $K$. Folse and Nowak (1995) also presented examples that included the effect of leaning columns.

The table below gives a comparison of the equivalent length method and the direct analysis method and describes how each method addresses the stability analysis requirements of AISC 360-10.
Table 3-3 Comparison of Direct Analysis Method and Equivalent Length Method (Adapted from Nair 2009)

<table>
<thead>
<tr>
<th>Comparison of Basic Stability Requirements with Specific Provisions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic Requirement in Section 1 of This Model Specification</td>
</tr>
<tr>
<td>(1) Consider second-order effects (both P-Δ and P-δ)</td>
</tr>
<tr>
<td>(2) Consider all deformations</td>
</tr>
<tr>
<td>(3) Consider geometric imperfections which include joint-position imperfections* and member imperfections</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>(4) Consider stiffness reduction due to inelasticity which affects structure response and member strength</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>(5) Consider uncertainty in strength and stiffness which affects structure response and member strength</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

* In typical building structures, the "joint-position imperfections" are the column out-of-plumbness.

** Second-order effects may be considered either by rigorous second-order analysis or by amplifications of the results of first-order analysis (using the B1 and B2 amplifiers in the AISC Specification).
3.5.5 **First Order Method**

The first order analysis method is a simplified method derived from the direct analysis method. (Kuchenbecker et al., 2004) The main benefit of this method is that only a first order analysis is required. In addition, because it is based on the direct analysis method, $K$ can be set at 1 for all cases. (AISC 360-10)

The main simplification in this method comes from the assumption that the ratio of second order drift to first order drift or $B_2$, is assumed to be equal to 1.5. From this assumption, equivalent notional lateral loads can be back calculated which simulate the effects of second order effects as well as reduction in stiffness due to partial yielding which are both accounted for using the direct analysis method. (Ziemian, 2010)

The simplification and assumptions which are made in development of the first order method lead to limitations on use of this method. The ratio of second order drift to first order drift or $B_2$ is assumed to be 1.5, which sets the maximum allowed value for application of this method. The stiffness reduction of 0.8 was also assumed in calculations of additional notion loads and therefore, based on the previous discussion of the reduction in stiffness, $\alpha_P P_y \leq 0.5$ must be maintained for the 0.8 reduction in stiffness to remain true.

The notional load $N_i$ for the first order analysis method is defined as: (Kuchenbecker et al., 2004)
\[
N_i = \left( \frac{B_2}{1 - 0.2B_2} \right) \frac{\Delta}{L} Y_i \geq \left( \frac{B_2}{1 - 0.2B_2} \right) 0.002Y_i
\]

If the above assumptions of \(B_2 = 1.5\) and \(\tau_b = 1.0\) are made and substituted into the equation above, it can be simplified to the form seen in AISC 360-10

\[
N_i = 2.1(\Delta/L)Y_i \geq 0.0042Y_i
\]

With:

- \(N_i\) = Notional load applied at level \(i\), kips
- \(\alpha\) = 1.0 (LRFD); 1.6 (ASD)
- \(Y_i\) = Gravity load applied at level \(i\) from the LRFD or ASD load combinations as applicable, kips
- \(\Delta\) = First order interstory drift
- \(L\) = Height of story

Unlike the effective length and direct analysis methods, the notional load is required to be added to all load combinations regardless of gravity only or lateral load combinations.

While this method has been simplified to eliminate the need for a second order analysis and any calculation of \(K\), the limitations and verification can hinder the use of this method. The simplifications made in the development of this method make it a very useful tool when frame analysis is done by hand because a second order analysis is not required. Most pipe rack structures are designed using modern
analysis software which is capable of performing a second order analysis, therefore the first order method will likely see limited use for pipe rack applications.
4. **Research Plan**

   A literature review was first conducted to gather and review the available information pertaining to the design and engineering of pipe rack structures for use in industrial facilities. The industry is constantly evolving, and the most current literature discussing the design and engineering of pipe racks was targeted for review. Next, a literature review was conducted to gather and review the available information pertaining to AISC stability analysis. Literature that described the various methods used by both AISC 360-05 and AISC 360-10 were the focus of the review. The direct analysis method literature was of particular interest as it was a relatively new development in regards to stability analysis.

   A general plan for the research that was conducted is presented here and is described as follows:

1. Use Benchmark Problems from AISC 360-10 to test the second order analysis capabilities of STAAD and report on the validity of the STAAD approach.
2. Describe in detail a typical pipe rack to be used for comparison of the methods.
3. Develop general loads and load combinations for use in the analysis models.
4. Develop a general STAAD.Pro V8i model that can be used for analysis of the Equivalent Length Method, Direct Analysis Method and First Order Method with input from [2] and [3].

5. Complete a first order analysis of the pipe rack structure developed in [4] for use in calculation of the $\Delta_2/\Delta_1$ ratio as well as for use in the First Order Method and discuss the results and validity of the method based on AISC limitations.


7. Optimize the strength only design of the test pipe rack structure developed in [4] using the Direct Analysis Method and compare the results to the Equivalent Length Method.

8. Use the models developed in [6] and [7] and vary member sizes and base fixity based on the serviceability limits and compare the results.

9. Compare the results of [5 to 8].
5. **Member Design**

The available strength of members should be calculated in accordance with the provisions of AISC 360-10 Chapters D, E, F, G, H, I, J and K. These chapters should be used regardless of the method of stability analysis chosen. Column design can become a major point of focus during stability analysis based on several factors. The effective length factor for column design can become complex for even simple structures when using the effective length method of stability analysis. Columns typically experience combined flexural, shear and axial load and the interaction between each stress must be investigated.

The two methods of stability analysis; effective length and direct analysis, as expected, produce varying load effects. A design example is shown in Figure 5-1 showing a simple cantilever W10X60 column bent about the strong axis with an axial load P and the horizontal load H = 10%P. The column is assumed to be supported out of plane resulting in only strong axis buckling. Stability analyses using both the equivalent length method and the direct analysis method were performed. A linear elastic analysis was performed as well to establish a first order baseline for comparison. STAAD.Pro V8i was used for both methods of stability analysis while hand calculations were done to compute the linear elastic forces.
The cantilever column was then checked against the code specified available strength. The column in this example will experience both axial and flexural loads, therefore AISC 360-10 Chapter H will be used to determine the available strength.

A simple cantilever column was chosen to simplify the selection of the effect length factor $K$. The theoretical value of $K$ for a fixed base cantilever column is 2. Therefore the effective length of the column used for the determining the available strength using the effective length method will be 30 ft.
Figure 5-2 shows a graph of axial load vs. moment for the simple cantilever example. AISC 360-10 equations H1-1a and H1-1b are included on the graph for design purposes. When comparing methods of analysis, with the linear elastic method as a baseline, the differences are readily apparent. As expected, the linear elastic method produces linear results of axial load vs. moment. Based on the theory of stability analysis, the linear elastic method is expected to produce results that overestimate the axial resistance and underestimate the moment demand. When comparing the results of both the effective length and direct analysis methods, the resistance based on AISC 360-10 equations H1-1a and H1-1b must be adjusted based on the effective length factor K. This will affect the axial resistance of the column section. The direct analysis method assumes that K = 1 and the effective length method assumes that K = 2 for a fixed base cantilever column. This changes the “anchor point” interaction equations.
Figure 5-2 Simple Cantilever Design Example Results

One of the main differences in results between the effective length method and the direct analysis method is the moment demand. The axial load resistance is comparable between the two methods but the moment demands can differ significantly. The column capacity is adjusted between the methods based on the effective length factor which calibrates the axial capacity. The difference in moment demand is due to the reduction in stiffness used in the direct analysis which increases deformations which in turn increases eccentricities and therefore moment demand is increased.
Based on the results of this design example, several observations can be made:

(AISC 360-10)

1. Accurate calculations of the effective length factor, K, are critical to achieving accurate results using the effective length method.

2. The moment demand is underestimated when using the effective length method. This can significantly affect the design loads for beams and connections which provide rotational resistance for the column.
6. **Pipe Rack Analysis**

6.1 **Generalized Pipe Rack**

A typical pipe rack will be developed and used for comparison purposes for this thesis. The typical pipe rack was chosen and modeled based on idealized conditions. A width of 15 feet was chosen to allow one-way traffic along the pipe rack corridor. The height of the first level of the pipe rack was set at 20 feet to provide sufficient height clearance along the access corridor.

The overall length of the pipe rack was set at 100 feet. Longer pipe rack sections are typically broken into shorter segments (100 to 200 feet) with each shorter segment separated by expansion joints to allow thermal expansion or contraction between segments. One of the lengthwise central bays of each segment is typically braced in the longitudinal direction. This allows the length of the pipe rack to expand and contract about a central braced bay and reduces thermally induced loads cause from restraint of thermal movement. If each end of the segment were to consist of a braced frame, the length of the pipe rack would essentially be locked in place and higher thermally induced loads would be seen.

Moment frames are typically spaced at 15-20 feet. This spacing is typically chosen based on the maximum allowable spans for the pipes or cable trays being supported. This spacing can vary based on the estimated size and allowable deflection limits of the pipe being supported.
Longitudinal struts are usually offset from the beams used to support the pipes. Levels of the pipe rack are assumed to be fully loaded with pipe, and when the pipes need to exit the rack to the side to connect to equipment, a flat turn cannot be used as this would clash with the other pipes on the same level. The pipe is typically routed to turn either up or down and then out of the rack at the level of the longitudinal struts where the pipe can be supported on the longitudinal struts before exiting the rack.

To allow room for pipes to enter and exit the pipe rack, a spacing of 5 feet between levels is typically used. If the pipe rack carries larger pipes, additional room may be required between levels. This spacing should be determine with the help of the piping engineer on the project. Spacing between pipe rack levels of 5 feet will be used in this thesis.

Figure 6-1 shows an isometric view of the typical pipe rack that will be used for analysis and comparison of stability analysis methods. While this is not representative of all pipe racks, it will still provide a useful basis for comparison purposes in pipe rack structures.
To simplify the design and analysis, a typical moment frame will be selected and isolated for analysis and design. Figure 6-2 shows an elevation view of a typical moment frame.
Out-of-plane supports were added at the locations of longitudinal struts which will restrain any movement in and out of the page (see Figure 6-1). Longitudinal struts all tie into the braced bay, therefore relatively small deflections will be experienced in the weak axis of the columns and the restraint of any movement in this direction is a reasonable assumption.

Based on initial calculations that compare the results of the isolated moment frame and the entire pipe rack segment, relatively small differences were seen. Because the braced bay supports any longitudinal loading, relatively very little weak axis column moment or longitudinal deflection occurs that would affect the design of
the columns or beams that are part of the moment frame. Ratios of demand to
capacity showed errors of less than 5% on member design when using the single
frame compared to the full pipe rack structure. Therefore, analysis of a single
moment frame will be used to simplify calculations. The focus of this thesis will
mainly be on the analysis of the moment frame; the braced frame will not be
considered in analysis and design.

In actual design, engineering judgment should be used to determine if the
analysis of a single frame simplification can be made. In many cases, pipe racks are
not symmetric and loading can vary from frame to frame and the overall structure
should be analyzed as a whole to determine the load effects. Bracing systems should
also be designed to resist the longitudinal loads of the entire segment and therefore
modeling of the entire pipe rack segment may be required to determine load path and
design loads for struts and braces.

6.2 **Pipe Rack Loading**

The pipe rack used for analysis and comparison of methods will have
consistent loading between methods to limit the number of variables. In general,
loads in the longitudinal direction will not be considered in design and comparison
because the focus of analysis will be on the behavior and performance of the moment
frame. Therefore, wind and seismic loading, will only be considered in the transverse
directions.
In practice, engineering judgment should be used in determining all applicable loads. The following discussion is meant to define loads only for analysis and comparison of the stability analysis and therefore certain simplifications are made to facilitate analysis but still provide results that are typical of pipe racks.

All loads are developed based on the assumption that the pipe rack is considered an Occupancy Category III. (PIP STC01015) This will affect the importance factor used for development of wind and seismic loading.

The primary load cases, which were defined previously, were chosen according to PIP STC01015. Additional primary load cases may be required based on the requirements of AISC 360-10 for notional loads. Loads are developed based in input from ASCE 7-05. While ASCE 7-10 is available, PIP STC01015 has not been updated to reflect the changes made by ASCE 7-10.

The first primary load case defined was the dead load of the structure ($D_s$). This is considered as the self-weight of the steel. For design, an additional 10% of the self-weight was added to account for fabrication tolerances and additional materials used for connections. No other loads were assumed at this time for the dead load of the structure.

The operating dead load as discussed previously is typically applied at 40 psf, which assumes a fully loaded level with 8 inch pipes full of water spaced at 15 inches. The representative pipe rack will use 40 psf applied over the entire tributary
area of the pipe rack. The spacing of the moment frames was chosen as 20 feet, therefore the uniform load applied at each level of the pipe rack is 800 pounds per foot. This load can be seen in Figure 6-3.

![Figure 6-3 Section View of Moment Frame - Operating Dead Load](image)

The empty dead load of the pipe can be taken as 60% of the operating dead load unless further information is known. This calculates to 480 pounds per foot. The empty dead load of the pipe is applied in a similar fashion as seen in Figure 6-3.

Test dead load is the weight of the pipe plus the weight of the test medium. This type of loading will typically control when the majority of the pipes in the pipe
rack are filled with gas or steam during operation. Hydro-testing is typically done to test the piping prior to startup. Pipes were assumed to be full of water for the operating load, therefore for this analysis, the test dead load is equivalent to the operating dead load and the load is exactly as seen in Figure 6-3.

The erection dead load can account for any additional loads or reduction in loads due to erection activities. This is typically used for any equipment and is based on the fabricated weight. For piping, the erection dead load and the empty dead load are typically the same. Therefore, the erection dead load case is not defined at this time and if required in load combinations, the empty dead load can be used in place of the erection dead load.

Pipe anchor and pipe friction loads are typically based on actual loading conditions of pipes located in the pipe rack. However, without final pipe loading, an estimate of pipe loads must be made. Friction forces can be estimated based on the coefficient of friction between the pipe shoe and support beam. This coefficient of friction is usually assumed to be 0.4. Application of 40% of the operating dead load tends to be extremely conservative. Friction loads are cause by expansion and contraction of pipes. Based on the expansion and contraction of pipes, friction loads are typically seen in the longitudinal direction of the pipe rack. Because it is highly unlikely that all pipes will expand and contract simultaneously and some pipes may contract while others expand, a more realistic value of 10% of operating dead load will be applied in the longitudinal direction. Friction loads are applied to the pipe
rack because the location of the loading could cause additional second order effects in the beams in the moment frames.

While friction loads and anchor loads are typically only seen in the longitudinal direction of the pipe rack, cases where anchor loads are seen in the transverse direction could happen. Therefore apply 5% of the operating dead load as a conservative estimate for the representative pipe rack. Figure 6-4 shows the application of pipe anchor loads. In practice, local members should be checked for pipe anchor loads and friction loads as the individual member design may be controlled by high anchor loads.
Self-straining thermal loads will not be considered in design. A design ΔT of 0 degrees Fahrenheit will be applied to all members. Thermal loading can cause problems if members are restrained from expansion or contraction. As the 2-D moment frame has very little resistance for thermal movements, thermal loads will not be considered in design of the representative model. Additional thermal considerations could be considered but are outside the scope of this thesis.
Live load is typically only applicable to platforms or walkways required for access, therefore live load will not be applied based on the assumption of no access platforms or walkways on the simplified pipe rack.

Similarly, snow load typically will not control the design and therefore will not be considered in the simplified analysis model.  (PIP STC01015)

Wind load is applied consistent with ASCE 7-05 principles. For comparison purposes, a 3 second gust wind velocity was assumed to be 100 miles per hour which should cover a larger majority of sites. Additional information given by ASCE Wind Loads for Petrochemical Facilities is included in wind load development. The following velocity pressures at specified heights was developed according to ASCE 7-05. Table 6-1 shows velocity pressures for cable tray and structural members and Table 6-2 shows velocity pressures for pipes.

### Table 6-1 Velocity pressures for cable tray and structural members

<table>
<thead>
<tr>
<th>Height (ft)</th>
<th>( q_z ) (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-15</td>
<td>21.270</td>
</tr>
<tr>
<td>20</td>
<td>22.522</td>
</tr>
<tr>
<td>25</td>
<td>23.523</td>
</tr>
<tr>
<td>30</td>
<td>24.524</td>
</tr>
<tr>
<td>40</td>
<td>26.025</td>
</tr>
</tbody>
</table>
Table 6-2 Velocity pressures for pipe

<table>
<thead>
<tr>
<th>Pipe</th>
<th>Height (ft)</th>
<th>( q_z ) (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-15</td>
<td>23.77</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>25.17</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>26.29</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>27.41</td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>29.09</td>
<td></td>
</tr>
</tbody>
</table>

The design wind force for structural members varies based on the projected area perpendicular to the wind direction and therefore will vary based on member size. In the case where the wind is parallel to the strong axis, the flange width defines the projected area. Structural shapes are assumed to have an average coefficient of drag \( C_D \) of 2.0 for all wind directions. Pipes were assumed to have a coefficient of drag \( C_D \) of 0.8 for wind perpendicular to pipe.

The design wind force for pipe is calculated based on the assumption of the largest pipe being 8 inches. 10% of the pipe rack width is added to the pipe added to the largest pipe diameter and multiplied by the bay spacing to determine a tributary area. (ASCE, 2011) To simplify the design, the total force from the pipes on each level is evenly divided and applied to each joint. This load could also be applied as a uniform load across the entire length of the beam. Table 6-3 shows the resulting joint loads resulting from wind loading on pipes.
Table 6-3 Resultant design wind force from pipe

<table>
<thead>
<tr>
<th>Height (ft)</th>
<th>F (lbs)</th>
<th>F/2 (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-15</td>
<td>701</td>
<td>350</td>
</tr>
<tr>
<td>20</td>
<td>742</td>
<td>371</td>
</tr>
<tr>
<td>25</td>
<td>775</td>
<td>387</td>
</tr>
<tr>
<td>30</td>
<td>808</td>
<td>404</td>
</tr>
<tr>
<td>40</td>
<td>857</td>
<td>429</td>
</tr>
</tbody>
</table>

Figure 6-5 shows the application of wind load for the typical pipe rack. The resultant load from pipe wind load is evenly distributed at the end joints. The design wind force shown for structural members is based on a column size of W10X33 with a flange width of 8 inches. This design wind force will vary based on column size but will provide a basis for application of wind load.
Seismic loading is based on specific site information. The ASCE 7-05 Equivalent Lateral Force Procedure will be used for seismic loading. Because this is a representative model, estimates on seismic loading will be made to simulate the general seismic load effects. The pipe rack is assumed to be located in site class C or below. An R value of 3 is chosen to simplify detailing requirements. The assumed seismic response coefficient, \( C_s \), will be 0.15. While the natural period of the structure will depend on member size and base support conditions, the natural period
will be assumed to be greater than 0.5 seconds which is used to determine the vertical
distribution.

Based on the above assumptions, seismic forces can be developed and applied
to the structure. The effective seismic mass is based on previously discussed dead
loads, both operating and empty dead loads. Table 6-4 shows the applied operating
seismic load applied at each level of the pipe rack while Table 6-5 shows the applied
empty seismic load. The lateral force at each level will be evenly divided and applied
to each joint. See Figure 6-6 for operating seismic loading. Empty seismic loading is
similar to loading shown in Figure 6-6

Table 6-4 Lateral seismic forces - operating

<table>
<thead>
<tr>
<th>Level</th>
<th>$F_x$ (lbs)</th>
<th>$F_x/2$ (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 1</td>
<td>950</td>
<td>475</td>
</tr>
<tr>
<td>Level 2</td>
<td>1450</td>
<td>725</td>
</tr>
<tr>
<td>Level 3</td>
<td>2100</td>
<td>1050</td>
</tr>
<tr>
<td>Level 4</td>
<td>2850</td>
<td>1425</td>
</tr>
</tbody>
</table>

Table 6-5 Lateral seismic forces - empty

<table>
<thead>
<tr>
<th>Level</th>
<th>$F_x$ (lbs)</th>
<th>$F_x/2$ (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 1</td>
<td>600</td>
<td>300</td>
</tr>
<tr>
<td>Level 2</td>
<td>900</td>
<td>450</td>
</tr>
<tr>
<td>Level 3</td>
<td>1250</td>
<td>625</td>
</tr>
<tr>
<td>Level 4</td>
<td>1700</td>
<td>850</td>
</tr>
</tbody>
</table>
Load combinations used for the representative model were developed from the PIP STC01015 load combinations as discussed in previous sections. As the model was simplified to isolate a moment resisting frame and study the results from essentially a 2-D analysis, loading in the transverse direction was the focus for
developing load combinations. Because lateral loads are typically reversible, this can create hundreds of load combinations if all load directions are considered. Transverse only loading simplified the loading and resulted in fewer load combinations. In the cases where notional loads are required, the notional load was considered to act only in the direction causing the worst effect on stability which is typically in the same direction as the lateral load.

Two sets of load combinations were developed for analysis and comparison purposes. The first set applied notional loads to only the gravity only load combinations. This set of load combinations is used for analysis using both the effective length and direct analysis methods. The first set of load combinations can only be used for the direct analysis when the ratio of second order to first order drift is less than 1.7 if using reduced stiffness or 1.5 using unreduced stiffness per AISC 360-10.

The second set of load combinations applies the notional loads as additive in all load combinations to create the worst effect on stability. The second set of load combinations was developed for analysis using the direct analysis method where the ratio of second order to first order drift is greater than 1.7 if using reduced stiffness or 1.5 using unreduced stiffness. The second set of load combinations can also be used for the first order method where the notional loads are additive for all load combinations. Although the magnitude of notional loads varies between the direct analysis and first order method, the notional loads can be adjusted in the primary load
combinations and the same load combinations can be used. Based on the limitation for use of the effective length method, the second set of load combinations with notional load applied in all load combinations is not required to be used in the effective length method.

A few observations on application of methods can be made by investigating the load combinations and requirements. First, in cases where the ratio of second order drift to first order drift is greater than 1.7 if using reduced stiffness or 1.5 using unreduced stiffness, the direct analysis method using the set of load combinations with additive notional loads in all load combinations is the suggested method of AISC 360-10. Next, all three methods can be used for analysis when the ratio of second order drift to first order drift is less than 1.7 if using reduced stiffness or 1.5 using unreduced stiffness. If this is the case, the direct analysis and effective length method do not require load combinations where the notional loads are additive in all cases, while the first order method requires additive notional loads in all load combinations.

Strength and serviceability will both be of interest in analysis and design, therefore both LRFD and ASD load combinations will be developed. LRFD load combinations will be used for member strength checks, while the serviceability checks will be made using the ASD load combinations. The load combinations used in analysis can be seen in Appendix 1 through 3.
6.4 **Strength and Serviceability Checks**

Strength and serviceability check are made using the capabilities of STAAD.Pro V8i. Strength checks are based on AISC 360-05. While AISC 360-10 has been released, STAAD.Pro V8i has yet to include the specification in design capabilities. Very few changes have been made in member capacity calculations and therefore the AISC 360-05 can be used for determining member capacity. The ratio of demand to capacity is a point of comparison between the various methods of stability analysis.

Serviceability checks are made using the calculated deflections from STAAD.Pro V8i. Unfactored loads combinations are used to calculated service deflections. Various limits on serviceability can be set based on specific project requirements.

AISC 360-10 states that both geometric imperfections and reduction in stiffness are not required in determining serviceability checks. Therefore, notional loads are not needed in service load combinations. Reduction in stiffness is also not included in calculations of deformations used for serviceability checks.

It should be noted that when using the direct analysis method in STAAD.Pro V8i, the reported deformations are calculated based on the reduced stiffness. AISC does not require the reduced stiffness to be used in serviceability checks, therefore the models should be analyzed using the unreduced stiffness for serviceability checks or
the reported deformations should be adjusted to account for the inclusion of reduced stiffness. Although not exact, reported deformations based on reduced stiffness could be multiplied by 0.8 to provide relatively accurate estimates of the actual deformations to be used in serviceability checks. This was based on several test models that were analyzed using both methods and the results were compared and found to be reasonable.

6.5 **Base Support Conditions**

Column base support conditions are affected by various factors. True fixed base columns, in actual conditions, can be very hard to achieve. Foundation types and anchor bolt layout and design can significantly affect the rotational resistance of the column base. Fixed base moment frames typically can see savings in member size but additional considerations in foundation and anchor bolt design could offset the savings in member sizing. Fixed base moment frames will also typically see a reduction in deformations due to the additional moment capacity generated by the base fixity. Pinned base moment frames on the other hand will typically require heavier members and experience potentially larger deformations compared to similar fixed base moment frames. Base support conditions can have a significant effect on overall frame behavior, therefore both fixed and pinned conditions were analyzed.

Base support conditions also become important in the calculation of the effective length factor (K) used in the effective length method. The alignment charts
from AISC 360-10 are based on the value $G$ which is a function of the rotational resistance provided by end constrains. For pinned conditions, $G$ is theoretically infinity. However, unless the connection is design as a true friction-free pin, $G$ should be assumed to be 10 for use in practical design. Unless true pin connections are used, this recognizes that pin connections have some moment resistance which affects the effective length factor. On the other hand, rigid connections have a theoretical $G$ value of 0, but for practical design, a value of 1 should be used. (AISC 360-10) This again recognizes that rigid or moment connections are not completely rigid and can have a slight rotation before full rigidity is reached. Based on AISC 360-10 recommendation, $G$ is assumed to equal 10 for the pin column support condition and 1 for the fixed column support condition. Additional modification in the calculation of the effective length factor, $K$, will be discussed in further sections.

6.6 Effective Length Factor

Many papers and books have extensive discussions on the calculation of the effective length factor $K$. The previous discussion of this topic in the Literature Review section provided additional sources and information for calculation of $K$. As the representative pipe rack structure used for analysis is a relatively straightforward moment resisting frame, the standard AISC 360-10 determination of $K$ can be used. However, AISC 360-10 does suggest a few adjustments when using the alignment charts. The adjustments are recommended based on the fact that the alignment charts
are developed based on the previously discussed assumptions. Adjustments made based on the column end conditions was discussed in the previous section.

Additional adjustments must be made based girder end connections, significant axial loads in girders, column inelasticity, and connection flexibility. (AISC 360-10)

For pinned base support conditions the following adjustments were used:

- $G = 10$ for pinned base column condition.
- $EI/L$ for girders was multiplied by $2/3$ to account for fixed end girders.
- $EI$ for columns was reduced to $0.8EI$ to account for column inelasticity.

Small axial loads were assumed in the girders, therefore no adjustments were made based on the axial load present in the girders.

$K$ values were calculated using the alignment charts and the equations for $G$ from AISC 360-10. For pinned base support conditions the following values were determined. See Figure 6-7 for $K$ values of each column section. $K$ values shown are based on W10X49 columns and W10X33 girders. For models where the member sizes were changed, $K$ values were recalculated based on actual members.
For the fixed base column condition, all the same adjustments were made except for the adjustment for column end condition. For the fixed base condition, \( G \) was set equal to 1. See Figure 6-8 for \( K \) values for each column section. \( K \) values shown are based on W10X33 columns and W10X26 girders. For models where the member sizes were changed, \( K \) values were recalculated based on actual members.

Figure 6-7 Effective Length Factor \( K \) – Pinned Base
Notional loads must be developed for use of the first order method. Notional loads used in the first order method account for second order effects, geometric imperfections and reduction in stiffness. Notional load were developed based on the following AISC 360-10 equation as defined previously:
\[ N_i = 2.1(Δ/L)Y_i \geq 0.0042Y_i \]

Notional loads based on the above equation must be developed based on the drift ratio at strength load levels. Initial target drifts may be calculated and therefore be used in the calculation of notional loads. These notional loads are then applied and the strength level drifts are verified to be less than the targeted drifts. The closer the strength level drift is to the target drift used in calculation of notional loads, the more accurate the results will be with this method.

Several iterations in the development of notional loads were completed to optimize and accurately apply the first order method. For the pinned base support condition, a target drift ratio \( Δ/L \) was set at \( h/65 \). Using the drift ratio of \( h/65 \), the notional load can be calculated to be \( 0.0323Y_i \) or 3.23\% of the gravity load at each level. It should be noted that this is significantly higher than the \( 0.002Y_i \) required for the effective length and direct analysis methods. Analysis based on the application of the previously calculated notional loads resulted in a strength level drift ratio of approximately \( h/67 \), therefore notional loads are correct. Additional calculations could be completed to determine notional loads which would provide more accurate results but the accuracy of the above target drift ratio is sufficient for comparison purposes.

Similarly, the target drift ratio for the fixed base condition was set at \( h/175 \). This drift ratio calculates to \( 0.012Y_i \) or 1.2\% of the gravity load at each level. Analysis based on the application of the previously calculated notional loads resulted
in a strength level drift ratio of approximately $h/182$. Again, additional calculations could be completed which provide more accurate results. However, the initial target drift ratio is sufficient for comparison purposes.

Notional load used in the first order method are additive in all load combinations. Load combinations were developed to apply these notional loads in all load combinations to create the most destabilizing effect. See Appendix 3 for STAAD input file for the representative model using the first order method.

6.8 STAAD Benchmark Validation

Some analysis software packages are capable of performing a rigorous second order analysis which can be used in any method that requires the inclusion of second order effects, such as the direct analysis method or equivalent length method. AISC 360-05 and 360-10 both give two benchmark problems to determine if the analysis procedure meets the requirements of a rigorous second order analysis. The benchmark problems can be seen in Figure 6-9.
STAAD.Pro V8i was used for modeling and analysis of pipe racks. To verify second order analysis capabilities in STAAD.Pro V8i, both benchmark problems from AISC 360-10 were run with the results listed in Table 6-6. AISC 360-10 specifies that moment corresponding to all axial load cases should agree within 3% and deflections within 5%. When comparing the solutions, it can be seen that the STAAD results have less than 0.5% variance from the AISC solutions. (AISC 360-10) Most of these small differences can probably be explained by rounding errors or precision of reported solutions. Therefore, STAAD.Pro V8i can be assumed to be correctly carrying out a rigorous second order analysis and therefore can be used to
analyze pipe racks using both the direct analysis method and equivalent length method.

### Table 6-6 Benchmark solutions

#### Benchmark Problem 1

<table>
<thead>
<tr>
<th>Axial Load, $P$ (kip)</th>
<th>$\Delta_{\text{mid}}$ (in)</th>
<th>$M_{\text{mid}}$ (kip*in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AISC Solution</td>
<td>STAAD Solution</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0.202</td>
<td>0.201</td>
</tr>
<tr>
<td>150</td>
<td>0.23</td>
<td>0.23</td>
</tr>
<tr>
<td>300</td>
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<td>0.268</td>
</tr>
<tr>
<td>450</td>
<td>0.322</td>
<td>0.322</td>
</tr>
</tbody>
</table>

#### Benchmark Problem 2

<table>
<thead>
<tr>
<th>Axial Load, $P$ (kip)</th>
<th>$\Delta_{\text{mid}}$ (in)</th>
<th>$M_{\text{mid}}$ (kip*in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AISC Solution</td>
<td>STAAD Solution</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>0.907</td>
<td>0.905</td>
</tr>
<tr>
<td>100</td>
<td>1.34</td>
<td>1.339</td>
</tr>
<tr>
<td>150</td>
<td>1.77</td>
<td>1.765</td>
</tr>
<tr>
<td>200</td>
<td>2.6</td>
<td>2.594</td>
</tr>
<tr>
<td></td>
<td>AISC Solution</td>
<td>STAAD Solution</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>336</td>
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<td>856</td>
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<tr>
<td>601</td>
<td>855</td>
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</tr>
</tbody>
</table>
7. **Comparison of Results**

Both pinned base and fixed base support condition models were developed for analysis and comparison of the three methods of stability analysis. See Appendix 1 for STAAD.Pro V8i input file for pipe rack analysis using the effective length method. Appendix 2 and 3 contain similar inputs for the direct analysis and first order method respectively. The first model was analyzed with a pinned base column. The member sizes were chosen without regard to serviceability and picked only to satisfy the load demand. First order method, effective length method and direct analysis method were all applied to the model and the results compiled. A first order linear elastic analysis was completed to provide a benchmark for comparison and calculation of the ratio of second order drift to first order drift.

Table 7-1 shows the ratio of second order to first order drift ($\Delta_2/\Delta_1$) based on the comparison of the benchmark linear elastic analysis to the effective length method analysis. It should be noted that these maximum deflections are based on LRFD load combinations. See Appendix 1 and 2 for details of LRFD load combinations. The maximum $\Delta_2/\Delta_1$ ratio is calculated as 1.15.
Table 7-1 Ratio $\Delta_2/\Delta_1$ effective length method – pinned base

<table>
<thead>
<tr>
<th>LRFD Load Combination Number</th>
<th>Linear Elastic Analysis Maximum Deflection (inch)</th>
<th>Effective Length Method Maximum Deflection (inch)</th>
<th>$\Delta_2/\Delta_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>201</td>
<td>1.638</td>
<td>1.877</td>
<td>1.15</td>
</tr>
<tr>
<td>202</td>
<td>1.635</td>
<td>1.862</td>
<td>1.14</td>
</tr>
<tr>
<td>203</td>
<td>1.638</td>
<td>1.877</td>
<td>1.15</td>
</tr>
<tr>
<td>204</td>
<td>1.635</td>
<td>1.862</td>
<td>1.14</td>
</tr>
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<td>5.229</td>
<td>5.908</td>
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</tr>
<tr>
<td>206</td>
<td>2.423</td>
<td>2.726</td>
<td>1.13</td>
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<td>207</td>
<td>2.42</td>
<td>2.701</td>
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<td>5.802</td>
<td>1.11</td>
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<td>5.053</td>
<td>5.762</td>
<td>1.14</td>
</tr>
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<td>2.247</td>
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<td>1.13</td>
</tr>
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<td>211</td>
<td>2.244</td>
<td>2.522</td>
<td>1.12</td>
</tr>
<tr>
<td>212</td>
<td>5.049</td>
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</tr>
<tr>
<td>213</td>
<td>3.825</td>
<td>4.055</td>
<td>1.06</td>
</tr>
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<td>214</td>
<td>3.824</td>
<td>4.007</td>
<td>1.05</td>
</tr>
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<td>215</td>
<td>5.052</td>
<td>5.482</td>
<td>1.09</td>
</tr>
<tr>
<td>216</td>
<td>2.247</td>
<td>2.425</td>
<td>1.08</td>
</tr>
<tr>
<td>217</td>
<td>2.245</td>
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<td>5.05</td>
<td>5.389</td>
<td>1.07</td>
</tr>
<tr>
<td>219</td>
<td>2.19</td>
<td>2.301</td>
<td>1.05</td>
</tr>
<tr>
<td>220</td>
<td>2.188</td>
<td>2.284</td>
<td>1.04</td>
</tr>
<tr>
<td>221</td>
<td>0.002</td>
<td>0.002</td>
<td>1.00</td>
</tr>
<tr>
<td>222</td>
<td>0.002</td>
<td>0.002</td>
<td>1.00</td>
</tr>
<tr>
<td>223</td>
<td>2.153</td>
<td>2.42</td>
<td>1.12</td>
</tr>
<tr>
<td>224</td>
<td>2.15</td>
<td>2.399</td>
<td>1.12</td>
</tr>
</tbody>
</table>

Maximum $\Delta_2/\Delta_1 = 1.15$

A few other observations can be made from the results seen in Table 7-1. The limitation of $\Delta_2/\Delta_1$ for use of the first order method set by AISC 360-10 is 1.5. Therefore, for the representative pinned base pipe rack, the first order method is a valid method for stability analysis. Also, AISC 360-10 sets limitations for use of notional loads. Because the maximum $\Delta_2/\Delta_1$ is less than 1.5, notional load only need
be applied to the gravity only load combinations for use in the effective length method.

Table 7-2 shows the ratio of second order to first order drift ($\Delta_2/\Delta_1$) based on the comparison of the benchmark linear elastic analysis to the direct analysis method analysis. As expected, the ratio $\Delta_2/\Delta_1$ is slightly higher based on the reduction in stiffness. The benchmark first order linear elastic analysis for this comparison included a reduced stiffness used in analysis. The increase in the ratio $\Delta_2/\Delta_1$ seen in Table 7-2 shows that the reduction in stiffness can amplify the second order effects. The maximum ratio $\Delta_2/\Delta_1$ is 1.21. Because the ratio $\Delta_2/\Delta_1$ is less than 1.7 (reduced stiffness is used to calculate drift), notional load need only be applied in the gravity only load combinations. (AISC 360-10)
Both Table 7-1 and 7-2 show the importance of consideration of stability analysis in design for pinned base conditions. For the representative pinned base model, stability analysis can amplify the deformation by up to 21% for this specific model. Deformation may not always be the focus of analysis and design but when...
checking serviceability limits, stability analysis can increase deformations significantly when compared to an elastic first order analysis.

The first order method was performed on the same model but as the method name implies, only a first order analysis is done and therefore the ratio $\Delta_2/\Delta_1$ cannot be directly calculated based on the drifts alone. However, based on the results of the previous two analyses, the ratio $\Delta_2/\Delta_1$ will be well below the 1.5 limitation set by AISC 360-10. Therefore the first order method is a valid type of stability analysis for the representative pinned base pipe rack.

Demand to capacity for members should also be used when comparing the types of stability analysis methods. Maximum demand to capacity ratio for both column and beam design is shown in Table 7-3.

<table>
<thead>
<tr>
<th>Table 7-3 Maximum demand to capacity ratio – pinned base</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Column (10X49) Maximum Demand to Capacity Ratio</strong></td>
</tr>
<tr>
<td>Linear Elastic Analysis</td>
</tr>
<tr>
<td>---------------------------</td>
</tr>
<tr>
<td>0.665</td>
</tr>
<tr>
<td><strong>Beam (W10X33) Maximum Demand to Capacity Ratio</strong></td>
</tr>
<tr>
<td>Linear Elastic Analysis</td>
</tr>
<tr>
<td>---------------------------</td>
</tr>
<tr>
<td>0.834</td>
</tr>
</tbody>
</table>

The linear elastic analysis was included as a benchmark for comparison. The linear elastic analysis can be seen to underestimate the demand to capacity ratios of
members, sometimes significantly. When comparing the direct analysis method and the first order method, it can be seen that the demand to capacity ratio is slightly higher when using the first order method. This is to be expected since the first order method is a simplification of the direct analysis built on conservative assumptions which will envelope the design. The effective length method has slightly higher ratios for column design and slightly lower for beam design. As discussed in previous sections for the effective length method, the column strength equations are adjusted using K to account for reduction in stiffness, but the moment can be underestimated for beams and connections which resist column rotation. The actual demand forces are listed in Table 7-4.

<table>
<thead>
<tr>
<th></th>
<th>Column (W12X53) Maximum Forces</th>
<th>Beam (W12X40) Maximum Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Linear Elastic Analysis</td>
<td>First Order Method</td>
</tr>
<tr>
<td>Strong Axis Moment (kip*ft)</td>
<td>102.94</td>
<td>130.34</td>
</tr>
<tr>
<td>Axial Load (kip)</td>
<td>54.88</td>
<td>58.61</td>
</tr>
<tr>
<td></td>
<td>Beam (W12X40) Maximum Forces</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Linear Elastic Analysis</td>
<td>First Order Method</td>
</tr>
<tr>
<td>Strong Axis Moment (kip*ft)</td>
<td>97.02</td>
<td>112.42</td>
</tr>
<tr>
<td>Axial Load (kip)</td>
<td>5.46</td>
<td>5.74</td>
</tr>
</tbody>
</table>

Based on Table 7-3 and 7-4 good correlation can be seen between the methods. The demand to capacity ratios for each method show results that are
expected based on the theory used to develop each method. The member forces have slight variation between methods based on the slight differences required in analysis in the methods. All results show similar relationships between each method. It should be noted that varying geometry could have a significant effect on the ratio $\Delta_2/\Delta_1$ which could limit the use of either the first order method or effective length method. In general for the pinned base support condition, columns have relatively low axial demand when compared to the compression failure load $P_y$. Large moments are developed in both the columns and beams and therefore the majority of the member capacity is used to resist the moment demand.

Similar tables to those seen above were also developed based on the fixed base support condition which can be seen on the following pages.
<table>
<thead>
<tr>
<th>LRFD Load Combination Number</th>
<th>Linear Elastic Analysis Maximum Deflection (inch)</th>
<th>Effective Length Method Maximum Deflection (inch)</th>
<th>( \Delta_2/\Delta_1 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>201</td>
<td>0.687</td>
<td>0.72</td>
<td>1.05</td>
</tr>
<tr>
<td>202</td>
<td>0.682</td>
<td>0.715</td>
<td>1.05</td>
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<tr>
<td>203</td>
<td>0.687</td>
<td>0.72</td>
<td>1.05</td>
</tr>
<tr>
<td>204</td>
<td>0.682</td>
<td>0.715</td>
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<td>224</td>
<td>0.808</td>
<td>0.841</td>
<td>1.04</td>
</tr>
</tbody>
</table>

Maximum \( \Delta_2/\Delta_1 = \) 1.05

When comparing the fixed base ratio \( \Delta_2/\Delta_1 \) for the effective length method, it can be seen that the maximum value is 1.05. While this is slightly less than for the pinned base support condition model, it still shows the significance of stability analysis in design. Table 7-6 below shows the same ratio \( \Delta_2/\Delta_1 \) for the direct analysis with both the linear elastic and direct analysis using the reduced stiffness in
calculation of deformations. As discussed previously, it is expected that the ratio \( \Delta_2/\Delta_1 \) is slight higher based on the reduced stiffness. Both tables do show however that the representative model with fixed base satisfies all the requirements for stability analysis by any of the three methods.

Table 7-6 Ratio \( \Delta_2/\Delta_1 \) direct analysis method – fixed base

<table>
<thead>
<tr>
<th>LRFD Load Combination Number</th>
<th>Linear Elastic Analysis Maximum Deflection (inch) - Reduced Stiffness</th>
<th>Effective Length Method Maximum Deflection (inch)</th>
<th>( \Delta_2/\Delta_1 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>201</td>
<td>0.859</td>
<td>0.917</td>
<td>1.07</td>
</tr>
<tr>
<td>202</td>
<td>0.853</td>
<td>0.911</td>
<td>1.07</td>
</tr>
<tr>
<td>203</td>
<td>0.859</td>
<td>0.917</td>
<td>1.07</td>
</tr>
<tr>
<td>204</td>
<td>0.853</td>
<td>0.911</td>
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<td>2.532</td>
<td>2.678</td>
<td>1.06</td>
</tr>
<tr>
<td>209</td>
<td>2.685</td>
<td>2.85</td>
<td>1.06</td>
</tr>
<tr>
<td>210</td>
<td>1.218</td>
<td>1.291</td>
<td>1.06</td>
</tr>
<tr>
<td>211</td>
<td>1.212</td>
<td>1.286</td>
<td>1.06</td>
</tr>
<tr>
<td>212</td>
<td>2.68</td>
<td>2.845</td>
<td>1.06</td>
</tr>
<tr>
<td>213</td>
<td>1.802</td>
<td>1.85</td>
<td>1.03</td>
</tr>
<tr>
<td>214</td>
<td>1.8</td>
<td>1.847</td>
<td>1.03</td>
</tr>
<tr>
<td>215</td>
<td>2.684</td>
<td>2.783</td>
<td>1.04</td>
</tr>
<tr>
<td>216</td>
<td>1.217</td>
<td>1.261</td>
<td>1.04</td>
</tr>
<tr>
<td>217</td>
<td>1.213</td>
<td>1.257</td>
<td>1.04</td>
</tr>
<tr>
<td>218</td>
<td>2.681</td>
<td>2.78</td>
<td>1.04</td>
</tr>
<tr>
<td>219</td>
<td>1.17</td>
<td>1.197</td>
<td>1.02</td>
</tr>
<tr>
<td>220</td>
<td>1.168</td>
<td>1.195</td>
<td>1.02</td>
</tr>
<tr>
<td>221</td>
<td>0.003</td>
<td>0.003</td>
<td>1.00</td>
</tr>
<tr>
<td>222</td>
<td>0.003</td>
<td>0.003</td>
<td>1.00</td>
</tr>
<tr>
<td>223</td>
<td>1.016</td>
<td>1.074</td>
<td>1.06</td>
</tr>
<tr>
<td>224</td>
<td>1.01</td>
<td>1.069</td>
<td>1.06</td>
</tr>
</tbody>
</table>

Maximum \( \Delta_2/\Delta_1 = \) 1.07
Table 7-7 Maximum demand to capacity ratio – fixed base

<table>
<thead>
<tr>
<th>Column (W10X33) Maximum Demand to Capacity Ratio</th>
<th>Linear Elastic Analysis</th>
<th>First Order Method</th>
<th>Effective Length Method</th>
<th>Direct Analysis Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.864</td>
<td>0.908</td>
<td>0.887</td>
<td>0.896</td>
<td></td>
</tr>
<tr>
<td>Beam (W10X26) Maximum Demand to Capacity Ratio</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.835</td>
<td>0.876</td>
<td>0.859</td>
<td>0.868</td>
<td></td>
</tr>
</tbody>
</table>

Table 7-8 Maximum demand forces – fixed base

<table>
<thead>
<tr>
<th>Column (W10X33) Maximum Forces</th>
<th>Linear Elastic Analysis</th>
<th>First Order Method</th>
<th>Effective Length Method</th>
<th>Direct Analysis Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strong Axis Moment (kip*ft)</td>
<td>60.136</td>
<td>63.35</td>
<td>62.35</td>
<td>63.19</td>
</tr>
<tr>
<td>Axial Load (kip)</td>
<td>46.03</td>
<td>46.83</td>
<td>46.42</td>
<td>46.56</td>
</tr>
<tr>
<td>Beam (W10X26) Maximum Forces</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strong Axis Moment (kip*ft)</td>
<td>61.1</td>
<td>64.12</td>
<td>62.91</td>
<td>63.56</td>
</tr>
<tr>
<td>Axial Load (kip)</td>
<td>5.12</td>
<td>5.23</td>
<td>5.15</td>
<td>5.15</td>
</tr>
</tbody>
</table>

In general, the first order method tends to be inherently more conservative based on the analysis assumptions which increase demand to account for the second order effects and reduction in stiffness. The effective length method on the other hand, tends to underestimate the demand while the capacity is adjusted to account for
the underestimation in demand by using the K factor. The direct analysis method will typically provide the most accurate results.

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

Serviceability limits were considered for additional analysis models. The representative pin based model was the focus based on large drift ratios when strength was the only consideration. A target serviceability limit was set at H/200. Members were resized based on this target and the results can be seen in the following tables.
<table>
<thead>
<tr>
<th>LRFD Load Combination Number</th>
<th>Linear Elastic Analysis Maximum Deflection (inch)</th>
<th>Effective Length Method Maximum Deflection (inch)</th>
<th>$\Delta_2/\Delta_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>201</td>
<td>0.837</td>
<td>0.897</td>
<td>1.07</td>
</tr>
<tr>
<td>202</td>
<td>0.834</td>
<td>0.892</td>
<td>1.07</td>
</tr>
<tr>
<td>203</td>
<td>0.837</td>
<td>0.897</td>
<td>1.07</td>
</tr>
<tr>
<td>204</td>
<td>0.834</td>
<td>0.892</td>
<td>1.07</td>
</tr>
<tr>
<td>205</td>
<td>2.807</td>
<td>2.987</td>
<td>1.06</td>
</tr>
<tr>
<td>206</td>
<td>1.374</td>
<td>1.46</td>
<td>1.06</td>
</tr>
<tr>
<td>207</td>
<td>1.372</td>
<td>1.451</td>
<td>1.06</td>
</tr>
<tr>
<td>208</td>
<td>2.805</td>
<td>2.96</td>
<td>1.06</td>
</tr>
<tr>
<td>209</td>
<td>2.581</td>
<td>2.758</td>
<td>1.07</td>
</tr>
<tr>
<td>210</td>
<td>1.149</td>
<td>1.224</td>
<td>1.07</td>
</tr>
<tr>
<td>211</td>
<td>1.146</td>
<td>1.217</td>
<td>1.06</td>
</tr>
<tr>
<td>212</td>
<td>2.579</td>
<td>2.733</td>
<td>1.06</td>
</tr>
<tr>
<td>213</td>
<td>2.09</td>
<td>2.155</td>
<td>1.03</td>
</tr>
<tr>
<td>214</td>
<td>2.089</td>
<td>2.142</td>
<td>1.03</td>
</tr>
<tr>
<td>215</td>
<td>2.581</td>
<td>2.691</td>
<td>1.04</td>
</tr>
<tr>
<td>216</td>
<td>1.148</td>
<td>1.194</td>
<td>1.04</td>
</tr>
<tr>
<td>217</td>
<td>1.147</td>
<td>1.188</td>
<td>1.04</td>
</tr>
<tr>
<td>218</td>
<td>2.579</td>
<td>2.668</td>
<td>1.03</td>
</tr>
<tr>
<td>219</td>
<td>1.119</td>
<td>1.148</td>
<td>1.03</td>
</tr>
<tr>
<td>220</td>
<td>1.118</td>
<td>1.143</td>
<td>1.02</td>
</tr>
<tr>
<td>221</td>
<td>0.001</td>
<td>0.001</td>
<td>1.00</td>
</tr>
<tr>
<td>222</td>
<td>0.001</td>
<td>0.001</td>
<td>1.00</td>
</tr>
<tr>
<td>223</td>
<td>1.177</td>
<td>1.249</td>
<td>1.06</td>
</tr>
<tr>
<td>224</td>
<td>1.174</td>
<td>1.242</td>
<td>1.06</td>
</tr>
</tbody>
</table>

Maximum $\Delta_2/\Delta_1 = 1.07$
Comparing the results of the pinned base (Tables 7-1 and 7-2) with the pinned base with serviceability limits imposed (Tables 7-9 and 7-10) shows an overall reduction in second order effects when serviceability controls the design. The overall structural stiffness is increased to limit deflections due to serviceability and therefore second order effects are minimized since the loads are acting on a less deformed shape.
Demand to capacity ratios show similar results when compared to previous results. Table 7-11 shows demand to capacity ratios when serviceability limits are imposed on the design. The demand to capacity ratios are much lower based on the additional stiffness required to meet serviceability limits.

**Table 7-11 Maximum demand to capacity ratio – pinned base serviceability limits**

<table>
<thead>
<tr>
<th>Column (W12X65)</th>
<th>Linear Elastic Analysis</th>
<th>First Order Method</th>
<th>Effective Length Method</th>
<th>Direct Analysis Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Demand to Capacity Ratio</td>
<td>0.418</td>
<td>0.452</td>
<td>0.483</td>
<td>0.447</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Beam (W12X45)</th>
<th>Linear Elastic Analysis</th>
<th>First Order Method</th>
<th>Effective Length Method</th>
<th>Direct Analysis Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Demand to Capacity Ratio</td>
<td>0.515</td>
<td>0.555</td>
<td>0.539</td>
<td>0.547</td>
</tr>
</tbody>
</table>

As seen in the results, both from the representative models and the cantilever column example, the effective length method tends to underestimate the moment demand in both the column and any beam or connection that resists column rotation. This fact should be considered in design if the effective length method is used for stability analysis.

The results also show the importance of stability analysis. Linear elastic analysis is not sufficient especially when designing moment frames. The results from above showed that the demand to capacity ratio could be underestimated by approximately 10-20% for the worst case when conducting a linear elastic analysis.
Varying the stiffness or geometry could easily produce greater errors in analysis if
stability analysis is not considered.
8. **Conclusions**

For the representative pipe rack model, both pinned and fixed base conditions, the first order, effective length, and direct analysis methods were all found to be valid methods of stability analysis according to AISC 360-10. When the ratios $\Delta_2/\Delta_1$ and $P_r/P_y$ are below the limits specified by AISC 360-10, all methods gave comparable results. Several observations on each method can be made based on the analysis and results.

The direct analysis method provides the most accurate results since reduction in stiffness is considered in analysis. There are also many benefits in application of this method with the most significant benefit being that the effective length factor $K$ can be set to 1 for all cases. This can significantly simplify calculations required to perform analysis especially for moment frames. Additionally, the direct analysis method has no limitations for use. Without limitations, no validation after analysis is required and therefore less time is required for this method. However, the best results from use of this method typically come from utilization of modern analysis software capable of performing rigorous second order analysis. Although an approximate second order analysis can be done by hand, this can become quite tedious and time consuming when other methods could produce similar results with less work.
The first order analysis is a mathematically simplified analysis method. The greatest benefit as the name implies, is that the method only requires a first order analysis. As with the direct analysis method, K can also be set at 1 for all cases. Based on the above simplifications, analysis can be significantly simplified. The simplified analysis tends to be the most beneficial for hand calculations of frames. The simplifications made in development of the method do however impose limitations on use. While the analysis portion may be less intensive, the post-analysis validation required can sometimes outweigh the benefits if the simplified analysis. Additionally, several iterations in analysis may be required to achieve the most accurate results since notional loads are calculated based on target drift limits. Application of notional load in all load combinations can also create significantly more load combinations which need to be considered in design. Simplifications in analysis also tend to create the most conservative results when compared to other methods of stability analysis.

The effective length method is probably the most well known method for stability analysis. However as the name implies, the effective length factor K must be calculated. Based on the discussions in previous sections, this can become very complex even for relatively simple structures. The accuracy of the effective length method is critically linked to accurate calculation of the effective length factor. Various methods for determining K have been developed but the most widely known is still the alignment charts. When using the alignment charts from AISC 360-10 the
engineer should be aware of the assumptions made in the development of the charts and the limitations for use. AISC 360-10 has set limitations for use of this method. Verification of applicability of effective length method after analysis could limit the use of this method. Second order effects should be considered in design which can be done using either a rigorous or approximate analysis. Results from this method show that the moment demand for beams and connections that resist column rotation can be underestimated. The underestimation of moments in elements resisting column rotation is due to no reduction in stiffness considered in analysis. The engineer should be aware of the underestimation of moment demand when designing beams and connections with results found using the effective length method.

When comparing the pro’s and con’s of each method, some observations can be made. If modern software analysis is available, the direct analysis method will most likely require the least amount of work to achieve the most accurate results. Although the effective length method is the most well known, engineers should be aware of limitations and the tendency for underestimation of moments in results. The first order method can be a powerful method if certain limitations are met and slightly conservative results are acceptable. However, with modern analysis software, the first order method will likely see limited applicability unless calculations are completed by hand.

Stability analysis as it relates specifically to pipe rack structures provides several points of interest. Base support conditions have a relatively large influence on
results of stability analysis. Pinned base pipe racks could have large deformations which tend to produce larger second order effects. If fixed base support conditions can be developed, member strength can be further utilized before serviceability limits are reached. However, fixed base support conditions in many cases can be difficult to achieve.

If each method is applied appropriately to pipe rack structures, based on the results above, good correlation can be seen between the first order, effective length and direct analysis methods. However, AISC 360-10 limitation must be verified for use of the first order and effective length methods on a case by case basis. For the representative models used above, all three methods met AISC 360-10 requirements and were found acceptable for stability analysis.

Pipe racks moment frames tend to support relatively large lateral loads through flexural resistance. Because the majority of the member capacity is used to resist the lateral loads, little capacity is left to resist axial loads, therefore the member is sized based primarily on the flexural demand. Based on the lower axial demands, the additional reduction in stiffness used in the direct analysis method due to large axial demand typically is not required.

The representative pipe rack chosen for analysis was a moderately simple frame and calculation of the effective length factor was relatively straightforward. However, actual pipe racks may require more complex structures which can greatly complicate the calculation of the effective length factor. Based on the possibility for
complex effective length factor calculations, the direct analysis and first order method could be much less calculation intensive and provide more accurate results.

Based on the above results and observations, I recommend the direct analysis as the first choice in stability analysis for pipe racks. While both the effective length and first order method provide relatively accurate results as long their respective requirements are met the direct analysis provides the most accurate results and has no limitations for use. The direct analysis method can also be the simplest method to apply if modern software analysis is utilized as no front end calculations or post-analysis verification are required.
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American Society of Civil Engineers (ASCE). (2010) “Minimum design loads for buildings and other structures (ASCE 7-10).” American Society of Civil Engineers, Reston, VA.
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American Society of Civil Engineers (ASCE). (1997) “Guideline for seismic evaluation and design of petrochemical facilities.” American Society of Civil Engineers, Reston, VA.


Canada Department of Railways and Canals. (1919). “The Quebec bridge over the St. Lawrence River near the city of Quebec on the line of the Canadian National Railways (Report No. 1).” Canada Department of Railways and Canals, Ottawa Canada


Euler, L. (1744). “Methodus inveniendi lineas curvas….” Bousquet, Lausanne.


Appendix A – STAAD Input Pinned Base Analysis - Effective Length Method

STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 1-25-2012
ENGINEER NAME DAN
END JOB INFORMATION
*
*EQUIVALENT LENGTH METHOD
*PINNED BASE MODEL
*
INPUT WIDTH 79
*
*DEFINES JOINT LOCATIONS
UNIT FEET KIP
JOINT COORDINATES
1 0 0 0; 2 0 20 0; 3 0 25 0; 4 0 30 0; 5 0 35 0; 6 15 0 0; 7 15 20 0;
8 15 25 0; 9 15 30 0; 10 15 35 0;
*
*DEFINES MEMBERS
MEMBER INCIDENCES
1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 2 7; 6 3 8; 7 4 9; 8 5 10; 9 6 7; 10 7 8; 11 8 9;
12 9 10;
*
*DEFINE PROPERTIES OF STEEL
DEFINE MATERIAL START
ISOTROPIC STEEL
E 4.176e+006
POISSON 0.3
DENSITY 0.489024
ALPHA 6.5e-006
DAMP 0.03
END DEFINE MATERIAL
*
*
*DEFINE MEMBER SHAPE
MEMBER PROPERTY AMERICAN
1 TO 4 9 TO 12 TABLE ST W10X49
5 TO 8 TABLE ST W10X33
*
*
CONSTANTS
*SETS ALL MEMBERS TO BE STEEL
MATERIAL STEEL ALL
*
*DEFINES SUPPORT CONDITIONS
SUPPORTS
1 6 PINNED
2 TO 5 7 TO 10 FIXED BUT FX FY MX MY MZ
*
*DEFINES WIND VELOCITY PRESSURE QZ FOR APPLICATION
DEFINE WIND LOAD
TYPE 1
INT 0.02127 0.02127 0.02252 0.02352 0.02452 0.02602 HEIG 0 15 20 25 30 40
*  
*  
*  
***************
*PRIMARY LOAD CASES
***************
*  
LOAD 1 DEAD LOAD (DS) STEEL AND FIREPROOFING
*SELFWEIGHT OF THE STRUCTURE PLUS 10% FOR CONNECTIONS
SELFWEIGHT Y -1.1
*  
*  
*  
LOAD 2 OPERATING DEAD LOAD (DO)
*40 PSF UNIFORM LOAD OVER TRIBUTARY AREA
*40PSF*20FEET = 800 PLF = 0.8 KLF
MEMBER LOAD
5 TO 8 UNI GY -0.8
*  
*  
*  
LOAD 3 EMPTY DEAD LOAD (DE)
*60% OF OPERATING DEAD LOAD
MEMBER LOAD
5 TO 8 UNI GY -0.48
*  
*  
*  
LOAD 4 TEST DEAD LOAD (DT)
*40 PSF UNIFORM LOAD OVER TRIBUTARY AREA
*40PSF*20FEET = 800 PLF = 0.8 KLF
MEMBER LOAD
5 TO 8 UNI GY -0.8
*  
*  
*  
LOAD 5 WIND LOAD X DIRECTION (WX)
*APPLIES PRESSURE GRADIENT TO MEMBERS IN MODEL,
*FACTOR = (GUST * SHAPE FACTOR)
*G = 0.85, CD = 2.0
*WIND LOAD ON STRUCTURAL MEMBERS
*APPLIED BASED ON PROJECTED AREA
WIND LOAD X 1.7 TYPE 1 OPEN
*WIND ON PIPE OR CABLE TRAY
JOINT LOAD
2 7 FX 0.371
3 8 FX 0.387
4 9 FX 0.404
5 10 FX 0.429
*
*
*
LOAD 7 SEISMIC LOAD X DIRECTION (EX)
*SEISMIC LOAD APPLIED BASED ON VERTICAL DISTRIBUTION
JOINT LOAD
2 7 FX 0.475
3 8 FX 0.725
4 9 FX 1.05
5 10 FX 1.425
*
*
*
LOAD 14 SEISMIC LOAD Y DIRECTION (EY)
*PRIMARY LOADS 1 AND 2 MULTIPLIED BY 10%
*LOAD 1 DEAD LOAD (DS) STEEL AND FIREPROOFING SELFWEIGHT Y -0.11
*
*LOAD 2 OPERATING DEAD LOAD (DO)
MEMBER LOAD
5 TO 8 UNI GY -0.08
*
*
*
LOAD 9 PIPE FRICTION LOAD (FF)
*10% OF OPERATING DEAD LOAD APPLIED LONGITUDINAL MEMBER LOAD
5 TO 8 UNI GZ 0.08
*
*
*
LOAD 10 PIPE ANCHOR LOAD (AF)
*5% OF OPERATING DEAD LOAD APPLIED TRANSVERSE MEMBER LOAD
5 TO 8 UNI GX 0.04
*
*
*
LOAD 11 THERMAL EXPANSION FORCE (T)
*DELTA T OF 0 DEGREES APPLIED TO ALL STRUCTURAL MEMBERS TEMPERATURE LOAD
LOAD 15 NOTIONAL LOAD (N)
*LOAD 1 AND 2 MULTIPLIED BY 0.002 AND APPLIED LATERALLY
*LOAD 1 DEAD LOAD (DS) STEEL AND FIREPROOFING
*SELFWEIGHT OF THE STRUCTURE PLUS 10% FOR CONNECTIONS
SELFWEIGHT X 0.0022
*
*LOAD 2 OPERATING DEAD LOAD (DO)
*40 PSF UNIFORM LOAD OVER TRIBUTARY AREA
*40PSF*20FEET = 800 PLF = 0.8 KLF
MEMBER LOAD
5 TO 8 UNI GX 0.0016
*
*
LOAD 20 NATURAL PERIOD CALCULATION
*LOAD 1 DEAD LOAD (DS) STEEL AND FIREPROOFING
SELFWEIGHT x -1.1
*LOAD 2 OPERATING DEAD LOAD (DO)
MEMBER LOAD
5 TO 8 UNI Gx -0.8
*LOAD 3 EMPTY DEAD LOAD (DE)
MEMBER LOAD
5 TO 8 UNI Gx -0.48
modal calculation requested
*
*
*****************************************************************************
* ASD LOAD COMBINATIONS
*****************************************************************************
* DS+DO+FF+T+AF+L+S
*****************************************************************************
LOAD 101 DS+DO+FF+T+AF
REPEAT LOAD
1 1.0 2 1.0 9 1.0 11 1.0 10 1.0
*
LOAD 102 DS+DO-FF+T-AF
REPEAT LOAD
1 1.0 2 1.0 9 -1.0 11 1.0 10 -1.0
*
LOAD 103 DS+DO+FF-T+AF
REPEAT LOAD
1 1.0 2 1.0 9 1.0 11 -1.0 10 1.0
*
LOAD 104 DS+DO-FF-T-AF
REPEAT LOAD
1 1.0 2 1.0 9 -1.0 11 -1.0 10 -1.0
*
* DS+DO+AF+W

LOAD 105 DS+DO+AF+WX
REPEAT LOAD
1 1.0 2 1.0 10 1.0 5 1.0
*
LOAD 106 DS+DO-AF+WX
REPEAT LOAD
1 1.0 2 1.0 10 -1.0 5 1.0
*
LOAD 107 DS+DO+AF-WX
REPEAT LOAD
1 1.0 2 1.0 10 1.0 5 -1.0
*
LOAD 108 DS+DO-AF-WX
REPEAT LOAD
1 1.0 2 1.0 10 -1.0 5 -1.0
*

* DS+DO+AF+0.7E

LOAD 109 DS+DO+AF+0.7EX+0.7EY
REPEAT LOAD
1 1.0 2 1.0 10 1.0 7 0.7 14 0.7
*
LOAD 110 DS+DO-AF+0.7EX+0.7EY
REPEAT LOAD
1 1.0 2 1.0 10 -1.0 7 0.7 14 0.7
*
LOAD 111 DS+DO+AF-0.7EX+0.7EY
REPEAT LOAD
1 1.0 2 1.0 10 1.0 7 -0.7 14 0.7
*
LOAD 112 DS+DO-AF-0.7EX+0.7EY
REPEAT LOAD
1 1.0 2 1.0 10 -1.0 7 -0.7 14 0.7
*

* DS+DE+W

LOAD 113 DS+DE+WX
REPEAT LOAD
1 1.0 3 1.0 5 1.0
*
LOAD 114 DS+DE-WX
REPEAT LOAD
1 1.0 3 1.0 5 -1.0
*
* 0.9DS+0.6DO+AF+0.7E
**************************************************
LOAD 115 0.9DS+0.6DO+AF+0.7EX-0.7EY
REPEAT LOAD
1 0.9 2 0.6 10 1.0 7 0.7 14 -0.7
* 
LOAD 116 0.9DS+0.6DO-AF+0.7EX-0.7EY
REPEAT LOAD
1 0.9 2 0.6 10 -1.0 7 0.7 14 -0.7
* 
LOAD 117 0.9DS+0.6DO+AF-0.7EX-0.7EY
REPEAT LOAD
1 0.9 2 0.6 10 1.0 -0.7 14 -0.7
* 
LOAD 118 0.9DS+0.6DO-AF-0.7EX-0.7EY
REPEAT LOAD
1 0.9 2 0.6 10 -1.0 7 -0.7 14 -0.7
* 
**************************************************
* 0.9(DS+DE)+0.7EE
**************************************************
LOAD 119 0.9(DS+DE)+0.42EX-0.42EY
REPEAT LOAD
1 0.9 3 0.9 7 0.42 14 -0.42
* 
LOAD 120 0.9(DS+DE)-0.42EX-0.42EY
REPEAT LOAD
1 0.9 3 0.9 7 -0.42 14 -0.42
* 
**************************************************
* DS+DT+WP
**************************************************
LOAD 121 DS+DT+WPX
REPEAT LOAD
1 1.0 4 1.0 5 0.56
* 
LOAD 122 DS+DT-WPX
REPEAT LOAD
1 1.0 4 1.0 5 -0.56
* 
* 
* 
* 
**************************************************
* LRFD LOAD COMBINATIONS
**************************************************
* 1.4(DS+DO+FF+T+AF)
**************************************************
LOAD 201 1.4(DS+DO+FF+T+AF)
REPEAT LOAD
1 1.4 2 1.4 9 1.4 11 1.4 10 1.4
*
LOAD 202 1.4(DS+DO-FF+T-AF)
REPEAT LOAD
1 1.4 2 1.4 9 -1.4 11 1.4 10 -1.4
*
LOAD 203 1.4(DS+DO+FF-T+AF)
REPEAT LOAD
1 1.4 2 1.4 9 1.4 11 -1.4 10 1.4
*
LOAD 204 1.4(DS+DO+FF-T-AF)
REPEAT LOAD
1 1.4 2 1.4 9 -1.4 11 -1.4 10 -1.4
*
**************************************************
* 1.2(DS+DO+AF)+1.6W
**************************************************
LOAD 205 1.2(DS+DO+AF)+1.6WX
REPEAT LOAD
1 1.2 2 1.2 10 1.2 5 1.6
*
LOAD 206 1.2(DS+DO-AF)+1.6WX
REPEAT LOAD
1 1.2 2 1.2 10 -1.2 5 1.6
*
LOAD 207 1.2(DS+DO+AF)-1.6WX
REPEAT LOAD
1 1.2 2 1.2 10 1.2 5 -1.6
*
LOAD 208 1.2(DS+DO-AF)-1.6WX
REPEAT LOAD
1 1.2 2 1.2 10 -1.2 5 -1.6
*
**************************************************
* 1.2(DS+DO+AF)+1.0Eo
**************************************************
LOAD 209 1.2(DS+DO+AF)+1.0EX+1.0EY
REPEAT LOAD
1 1.2 2 1.2 10 1.2 7 1 14 1.0
*
LOAD 210 1.2(DS+DO-AF)+1.0EX+1.0EY
REPEAT LOAD
1 1.2 2 1.2 10 -1.2 7 1 14 1.0
*
LOAD 211 1.2(DS+DO+AF)-1.0EX+1.0EY
REPEAT LOAD
1 1.2 2 1.2 10 1.2 7 -1.0 14 1.0
*
LOAD 212 1.2(DS+DO-AF)-1.0EX+1.0EY
REPEAT LOAD
1 1.2 2 1.2 10 -1.2 7 -1.0 14 1.0
* 
**********************************************************************
* 0.9(DE+DE)+1.6W
**********************************************************************
LOAD 213 0.9(DS+DE)+1.6WX
REPEAT LOAD
1 0.9 3 0.9 5 1.6
* 
LOAD 214 0.9(DS+DE)+1.6WX
REPEAT LOAD
1 0.9 3 0.9 5 -1.6
* 
**********************************************************************
* 0.9(DS+DO)+1.2AF+1.0E
**********************************************************************
LOAD 215 0.9(DS+DO)+1.2AF+EX-EY
REPEAT LOAD
1 0.9 2 0.9 10 1.2 7 1.0 14 -1.0
* 
LOAD 216 0.9(DS+DO)+1.2AF+EX-EY
REPEAT LOAD
1 0.9 2 0.9 10 -1.2 7 1.0 14 -1.0
* 
LOAD 217 0.9(DS+DO)+1.2AF-EX-EY
REPEAT LOAD
1 0.9 2 0.9 10 -1.2 7 -1.0 14 -1.0
* 
LOAD 218 0.9(DS+DO)-1.2AF-EX-EY
REPEAT LOAD
1 0.9 2 0.9 10 1.2 7 -1.0 14 -1.0
* 
**********************************************************************
* 0.9(DS+DE)+1.0EE
**********************************************************************
LOAD 219 0.9(DS+DE)+EEX-EEY
REPEAT LOAD
1 0.9 3 0.9 7 0.6 14 -0.6
* 
LOAD 220 0.9(DS+DE)-EEX-EEY
REPEAT LOAD
1 0.9 3 0.9 7 -0.6 14 -0.6
* 
**********************************************************************
* 1.4(DS+DT)
**********************************************************************
LOAD 221 1.4(DS+DT+NX)
REPEAT LOAD
1 1.4 4 1.4
* LOAD 222 1.4(DS+DT-NX)
REPEAT LOAD
 1 1.4 4 1.4
*
**************************************************
* 1.2(DS+DT)+1.6(0.56W)
**************************************************
LOAD 223 1.2(DS+DT)+1.6(0.56WX)
REPEAT LOAD
 1 1.2 4 1.2 5 0.9
*
LOAD 224 1.2(DS+DT)-1.6(0.56WX)
REPEAT LOAD
 1 1.2 4 1.2 5 -0.9
*
*
*
PDELTA KG ANALYSIS
LOAD LIST 201 TO 224
PARAMETER 1
CODE AISC UNIFIED
FYLD 7200 ALL
*
KZ 2.7 MEMB 1 9
KZ 2.7 MEMB 2 10
KZ 3 MEMB 3 11
KZ 2.2 MEMB 4 12
KY 1 ALL
*
CHECK CODE ALL
*
FINISH
Appendix B – STAAD Input Pinned Base Analysis - Direct Analysis Method

STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 1-25-2012
ENGINEER NAME DAN
END JOB INFORMATION
*
*DIRECT ANALYSIS METHOD
*PINNED BASE MODEL
*
INPUT WIDTH 79
*
*DEFINES JOINT LOCATIONS
UNIT FEET KIP
JOINT COORDINATES
1 0 0 0; 2 0 20 0; 3 0 25 0; 4 0 30 0; 5 0 35 0; 6 15 0 0; 7 15 20 0;
8 15 25 0; 9 15 30 0; 10 15 35 0;
*
*DEFINES MEMBERS
MEMBER INCIDENCES
1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 2 7; 6 3 8; 7 4 9; 8 5 10; 9 6 7; 10 7 8; 11 8 9;
12 9 10;
*
*DEFINE PROPERITIES OF STEEL
DEFINE MATERIAL START
ISOTROPIC STEEL
E 4.176e+006
POISSON 0.3
DENSITY 0.489024
ALPHA 6.5e-006
DAMP 0.03
END DEFINE MATERIAL
*
*
*DEFINE MEMBER SHAPE
MEMBER PROPERTY AMERICAN
1 TO 4 9 TO 12 TABLE ST W10X49
5 TO 8 TABLE ST W10X33
*
*
CONSTANTS
*SETS ALL MEMBERS TO BE STEEL
MATERIAL STEEL ALL
* *DEFINES SUPPORT CONDITIONS
SUPPORTS
1 6 PINNED
2 TO 5 7 TO 10 FIXED BUT FX FY MX MY MZ
*
DEFINE DIRECT ANALYSIS
FYLD 7200 ALL
FLEX 1 ALL
AXIAL ALL
NOTIONAL LOAD FACTOR 0.002
END

*DEFINES WIND VELOCITY PRESSURE QZ FOR APPLICATION
DEFINE WIND LOAD
TYPE 1
INT 0.02127 0.02127 0.02252 0.02352 0.02452 0.02602 HEIG 0 15 20 25 30 40
*
*
*
**********************************
*PRIMARY LOAD CASES
**********************************
*
LOAD 1 DEAD LOAD (DS) STEEL AND FIREPROOFING
*SELFWEIGHT OF THE STRUCTURE PLUS 10% FOR CONNECTIONS
SELFWEIGHT Y -1.1
*
*
*
LOAD 2 OPERATING DEAD LOAD (DO)
*40 PSF UNIFORM LOAD OVER TRIBUTARY AREA
*40PSF*20FEET = 800 PLF = 0.8 KLF
MEMBER LOAD
5 TO 8 UNI GY -0.8
*
*
*
LOAD 3 EMPTY DEAD LOAD (DE)
*60% OF OPERATING DEAD LOAD
MEMBER LOAD
5 TO 8 UNI GY -0.48
*
*
*
LOAD 4 TEST DEAD LOAD (DT)
*40 PSF UNIFORM LOAD OVER TRIBUTARY AREA
*40PSF*20FEET = 800 PLF = 0.8 KLF
MEMBER LOAD
LOAD 5 WIND LOAD X DIRECTION (WX)
*APPLIES PRESSURE GRADIENT TO MEMBERS IN MODEL,
*FACTOR = (GUST * SHAPE FACTOR)
*G = 0.85, CD = 2.0
*WIND LOAD ON STRUCTURAL MEMBERS
*APPLIED BASED ON PROJECTED AREA
WIND LOAD X 1.7 TYPE 1 OPEN
*WIND ON PIPE OR CABLE TRAY
JOINT LOAD
2 7 FX 0.371
3 8 FX 0.387
4 9 FX 0.404
5 10 FX 0.429

LOAD 7 SEISMIC LOAD X DIRECTION (EX)
*SEISMIC LOAD APPLIED BASED ON VERTICAL DISTRIBUTION
JOINT LOAD
2 7 FX 0.475
3 8 FX 0.725
4 9 FX 1.05
5 10 FX 1.425

LOAD 14 SEISMIC LOAD Y DIRECTION (EY)
*PRIMARY LOADS 1 AND 2 MULTIPLIED BY 10%
*LOAD 1 DEAD LOAD (DS) STEEL AND FIREPROOFING
SELFWEIGHT Y -0.11
*LOAD 2 OPERATING DEAD LOAD (DO)
MEMBER LOAD
5 TO 8 UNI GY -0.08

LOAD 9 PIPE FRICTION LOAD (FF)
*10% OF OPERATING DEAD LOAD APPLIED LONGITUDINAL
MEMBER LOAD
5 TO 8 UNI GZ 0.08

LOAD 10 PIPE ANCHOR LOAD (AF)
*5% OF OPERATING DEAD LOAD APPLIED TRANSVERSE
MEMBER LOAD
5 TO 8 UNI GX 0.04

LOAD 11 THERMAL EXPANSION FORCE (T)
*DELTA T OF 0 DEGREES APPLIED TO ALL STRUCTURAL MEMBERS
TEMPERATURE LOAD
1 TO 12 TEMP 0

LOAD 15 NOTIONAL LOAD (N)
*LOAD 1 AND 2 MULTIPLIED BY 0.002 AND APPLIED LATERALLY
*LOAD 1 DEAD LOAD (DS) STEEL AND FIREPROOFING
*SELFWEIGHT OF THE STRUCTURE PLUS 10% FOR CONNECTIONS
SELFWEIGHT X 0.0022

*LOAD 2 OPERATING DEAD LOAD (DO)
*40 PSF UNIFORM LOAD OVER TRIBUTARY AREA
*40PSF*20FEET = 800 PLF = 0.8 KLF
MEMBER LOAD
5 TO 8 UNI GX 0.0016

LOAD 20 NATURAL PERIOD CALCULATION
*LOAD 1 DEAD LOAD (DS) STEEL AND FIREPROOFING
SELFWEIGHT X -1.1
*LOAD 2 OPERATING DEAD LOAD (DO)
MEMBER LOAD
5 TO 8 UNI Gx -0.8
*LOAD 3 EMPTY DEAD LOAD (DE)
MEMBER LOAD
5 TO 8 UNI Gx -0.48
modal calculation requested

* ASD LOAD COMBINATIONS
**************************************************
* DS+DO+FF+T+AF+L+S
**************************************************
LOAD 101 DS+DO+FF+T+AF
REPEAT LOAD
1 1.0 2 1.0 9 1.0 11 1.0 10 1.0

LOAD 102 DS+DO-FF+T-AF
REPEAT LOAD
1 1.0 2 1.0 9 -1.0 11 1.0 10 -1.0
LOAD 103 DS+DO+FF-T+AF
REPEAT LOAD
1 1.0 2 1.0 9 1.0 11 -1.0 10 1.0
*
LOAD 104 DS+DO-FF-T-AF
REPEAT LOAD
1 1.0 2 1.0 9 -1.0 11 -1.0 10 -1.0
*
**************************************************
* DS+DO+AF+W
**************************************************
LOAD 105 DS+DO+AF+WX
REPEAT LOAD
1 1.0 2 1.0 10 1.0 5 1.0
*
LOAD 106 DS+DO-AF+WX
REPEAT LOAD
1 1.0 2 1.0 10 -1.0 5 1.0
*
LOAD 107 DS+DO+AF-WX
REPEAT LOAD
1 1.0 2 1.0 10 1.0 5 -1.0
*
LOAD 108 DS+DO-AF-WX
REPEAT LOAD
1 1.0 2 1.0 10 -1.0 5 -1.0
*
**************************************************
* DS+DO+AF+0.7E
**************************************************
LOAD 109 DS+DO+AF+0.7EX+0.7EY
REPEAT LOAD
1 1.0 2 1.0 10 1.0 7 0.7 14 0.7
*
LOAD 110 DS+DO-AF+0.7EX+0.7EY
REPEAT LOAD
1 1.0 2 1.0 10 -1.0 7 0.7 14 0.7
*
LOAD 111 DS+DO+AF-0.7EX+0.7EY
REPEAT LOAD
1 1.0 2 1.0 10 1.0 7 -0.7 14 0.7
*
LOAD 112 DS+DO-AF-0.7EX+0.7EY
REPEAT LOAD
1 1.0 2 1.0 10 -1.0 7 -0.7 14 0.7
*
**************************************************
* DS+DE+W
**************************************************
LOAD 113 DS+DE+WX
REPEAT LOAD
1 1.0 3 1.0 5 1.0
*
LOAD 114 DS+DE-WX
REPEAT LOAD
1 1.0 3 1.0 5 -1.0
*
**************************************************
* 0.9DS+0.6DO+AF+0.7E
**************************************************
LOAD 115 0.9DS+0.6DO+AF+0.7EX-0.7EY
REPEAT LOAD
1 0.9 2 0.6 10 1.0 7 0.7 14 -0.7
*
LOAD 116 0.9DS+0.6DO-AF+0.7EX-0.7EY
REPEAT LOAD
1 0.9 2 0.6 10 -1.0 7 0.7 14 -0.7
*
LOAD 117 0.9DS+0.6DO+AF-0.7EX-0.7EY
REPEAT LOAD
1 0.9 2 0.6 10 1.0 7 -0.7 14 -0.7
*
LOAD 118 0.9DS+0.6DO-AF-0.7EX-0.7EY
REPEAT LOAD
1 0.9 2 0.6 10 -1.0 7 -0.7 14 -0.7
*
**************************************************
* 0.9(DS+DE)+0.7EE
**************************************************
LOAD 119 0.9(DS+DE)+0.42EX-0.42EY
REPEAT LOAD
1 0.9 3 0.9 7 0.42 14 -0.42
*
LOAD 120 0.9(DS+DE)-0.42EX-0.42EY
REPEAT LOAD
1 0.9 3 0.9 7 -0.42 14 -0.42
*
**************************************************
* DS+DT+WP
**************************************************
LOAD 121 DS+DT+WPX
REPEAT LOAD
1 1.0 4 1.0 5 0.56
*
LOAD 122 DS+DT-WPX
REPEAT LOAD
1 1.0 4 1.0 5 -0.56
*
LRFD LOAD COMBINATIONS

1.4(DS+DO+FF+T+AF)

LOAD 201 1.4(DS+DO+FF+T+AF)
REPEAT LOAD
1 1.4 2 1.4 9 1.4 11 1.4 10 1.4
*
LOAD 202 1.4(DS+DO-FF+T-AF)
REPEAT LOAD
1 1.4 2 1.4 9 -1.4 11 1.4 10 -1.4
*
LOAD 203 1.4(DS+DO+FF-T+AF)
REPEAT LOAD
1 1.4 2 1.4 9 1.4 11 -1.4 10 1.4
*
LOAD 204 1.4(DS+DO-FF-T-AF)
REPEAT LOAD
1 1.4 2 1.4 9 -1.4 11 -1.4 10 -1.4
*

1.2(DS+DO+AF)+1.6W

LOAD 205 1.2(DS+DO+AF)+1.6WX
REPEAT LOAD
1 1.2 2 1.2 10 1.2 5 1.6
*
LOAD 206 1.2(DS+DO-AF)+1.6WX
REPEAT LOAD
1 1.2 2 1.2 10 -1.2 5 1.6
*
LOAD 207 1.2(DS+DO+AF)-1.6WX
REPEAT LOAD
1 1.2 2 1.2 10 1.2 5 -1.6
*
LOAD 208 1.2(DS+DO-AF)-1.6WX
REPEAT LOAD
1 1.2 2 1.2 10 -1.2 5 -1.6
*

1.2(DS+DO+AF)+1.0E0

LOAD 209 1.2(DS+DO+AF)+1.0EX+1.0EY
REPEAT LOAD
1 1.2 2 1.2 10 1.2 7 1 14 1.0
*
LOAD 210 1.2(DS+DO-AF)+1.0EX+1.0EY
REPEAT LOAD
1 1.2 2 1.2 10 -1.2 7 1 14 1.0
* 
LOAD 211 1.2(DS+DO+AF)-1.0EX+1.0EY
REPEAT LOAD
1 1.2 2 1.2 10 1.2 7 -1.0 14 1.0
* 
LOAD 212 1.2(DS+DO-AF)-1.0EX+1.0EY
REPEAT LOAD
1 1.2 2 1.2 10 -1.2 7 -1.0 14 1.0
* 
**********************************************************************
* 0.9(DE+DE)+1.6W
**********************************************************************
LOAD 213 0.9(DS+DE)+1.6WX
REPEAT LOAD
1 0.9 3 0.9 5 1.6
* 
LOAD 214 0.9(DS+DE)-1.6WX
REPEAT LOAD
1 0.9 3 0.9 5 -1.6
* 
**********************************************************************
* 0.9(DS+DO)+1.2AF+1.0E
**********************************************************************
LOAD 215 0.9(DS+DO)+1.2AF+EX-EY
REPEAT LOAD
1 0.9 2 0.9 10 1.2 7 1.0 14 -1.0
* 
LOAD 216 0.9(DS+DO)-1.2AF+EX-EY
REPEAT LOAD
1 0.9 2 0.9 10 -1.2 7 1.0 14 -1.0
* 
LOAD 217 0.9(DS+DO)+1.2AF-EX-EY
REPEAT LOAD
1 0.9 2 0.9 10 1.2 7 -1.0 14 -1.0
* 
LOAD 218 0.9(DS+DO)-1.2AF-EX-EY
REPEAT LOAD
1 0.9 2 0.9 10 -1.2 7 -1.0 14 -1.0
* 
**********************************************************************
* 0.9(DS+DE)+1.0EE
**********************************************************************
LOAD 219 0.9(DS+DE)+EEX-EEY
REPEAT LOAD
1 0.9 3 0.9 7 0.6 14 -0.6
* 
LOAD 220 0.9(DS+DE)-EEX-EEY
REPEAT LOAD
1 0.9 3 0.9 7 -0.6 14 -0.6
*
************************************************************************
* 1.4(DS+DT)
************************************************************************
LOAD 221 1.4(DS+DT+NX)
REPEAT LOAD
1 1.4 4 1.4
*
LOAD 222 1.4(DS+DT-NX)
REPEAT LOAD
1 1.4 4 1.4
*
************************************************************************
* 1.2(DS+DT)+1.6(0.56W)
************************************************************************
LOAD 223 1.2(DS+DT)+1.6(0.56WX)
REPEAT LOAD
1 1.2 4 1.2 5 0.9
*
LOAD 224 1.2(DS+DT)-1.6(0.56WX)
REPEAT LOAD
1 1.2 4 1.2 5 -0.9
*
*
* PERFORM DIRECT ANALYSIS LRFD ITERATION 10 TAUTOL 0.01 DISPTOL 1e-014
LOAD LIST 201 TO 224
PARAMETER 1
CODE AISC UNIFIED
FYLD 7200 ALL
*
KZ 1 ALL
KY 1 ALL
*
CHECK CODE ALL
*
FINISH
Appendix C – STAAD Input Pinned Base Analysis - First Order Method

STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 1-25-2012
ENGINEER NAME DAN
END JOB INFORMATION
*
*FIRST ORDER METHOD
*PINNED BASE MODEL
*
INPUT WIDTH 79
*
*DEFINES JOINT LOCATIONS
UNIT FEET KIP
JOINT COORDINATES
1 0 0 0; 2 0 20 0; 3 0 25 0; 4 0 30 0; 5 0 35 0; 6 15 0 0; 7 15 20 0;
8 15 25 0; 9 15 30 0; 10 15 35 0;
*
*DEFINE MEMBER SHAPE
MEMBER INCIDENCES
1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 2 7; 6 3 8; 7 4 9; 8 5 10; 9 6 7; 10 7 8; 11 8 9;
12 9 10;
*
*DEFINE PROPERTIES OF STEEL
DEFINE MATERIAL START
ISOTROPIC STEEL
E 4.176e+006
POISSON 0.3
DENSITY 0.489024
ALPHA 6.5e-006
DAMP 0.03
END DEFINE MATERIAL
*
*
*DEFINE MEMBER SHAPE
MEMBER PROPERTY AMERICAN
1 TO 4 9 TO 12 TABLE ST W10X49
5 TO 8 TABLE ST W10X33
*
*
CONSTANTS
*SETS ALL MEMBERS TO BE STEEL
MATERIAL STEEL ALL
*
*DEFINES SUPPORT CONDITIONS
SUPPORTS
1 6 PINNED
2 TO 5 7 TO 10 FIXED BUT FX FY MX MY MZ
*
*DEFINES WIND VELOCITY PRESSURE QZ FOR APPLICATION
DEFINE WIND LOAD
TYPE 1
INT 0.02127 0.02127 0.02252 0.02352 0.02452 0.02602 HEIG 0 15 20 25 30 40
*
*
*************************************************************
*PRIMARY LOAD CASES
*************************************************************
* LOAD 1 DEAD LOAD (DS) STEEL AND FIREPROOFING
* SELFWEIGHT OF THE STRUCTURE PLUS 10% FOR CONNECTIONS
SELFWEIGHT Y -1.1
*
*
*
LOAD 2 OPERATING DEAD LOAD (DO)
* 40 PSF UNIFORM LOAD OVER TRIBUTARY AREA
* 40PSF*20FEET = 800 PLF = 0.8 KLF
MEMBER LOAD
5 TO 8 UNI GY -0.8
*
*
*
LOAD 3 EMPTY DEAD LOAD (DE)
* 60% OF OPERATING DEAD LOAD
MEMBER LOAD
5 TO 8 UNI GY -0.48
*
*
*
LOAD 4 TEST DEAD LOAD (DT)
* 40 PSF UNIFORM LOAD OVER TRIBUTARY AREA
* 40PSF*20FEET = 800 PLF = 0.8 KLF
MEMBER LOAD
5 TO 8 UNI GY -0.8
*
*
*
LOAD 5 WIND LOAD X DIRECTION (WX)
* APPLIES PRESSURE GRADIENT TO MEMBERS IN MODEL,
* FACTOR = (GUST * SHAPE FACTOR)
* G = 0.85, CD = 2.0
WIND LOAD ON STRUCTURAL MEMBERS
*APPLIED BASED ON PROJECTED AREA
WIND LOAD X 1.7 TYPE 1 OPEN
*WIND ON PIPE OR CABLE TRAY
JOINT LOAD
2 7 FX 0.371
3 8 FX 0.387
4 9 FX 0.404
5 10 FX 0.429
*
*
*
LOAD 7 SEISMIC LOAD X DIRECTION (EX)
*SEISMIC LOAD APPLIED BASED ON VERTICAL DISTRIBUTION
JOINT LOAD
2 7 FX 0.475
3 8 FX 0.725
4 9 FX 1.05
5 10 FX 1.425
*
*
*
LOAD 14 SEISMIC LOAD Y DIRECTION (EY)
*PRIMARY LOADS 1 AND 2 MULTIPLIED BY 10%
*LOAD 1 DEAD LOAD (DS) STEEL AND FIREPROOFING
SELFWEIGHT Y -0.11
*
*LOAD 2 OPERATING DEAD LOAD (DO)
MEMBER LOAD
5 TO 8 UNI GY -0.08
*
*
*
LOAD 9 PIPE FRICTION LOAD (FF)
*10% OF OPERATING DEAD LOAD APPLIED LONGITUDINAL
MEMBER LOAD
5 TO 8 UNI GZ 0.08
*
*
*
LOAD 10 PIPE ANCHOR LOAD (AF)
*5% OF OPERATING DEAD LOAD APPLIED TRANSVERSE
MEMBER LOAD
5 TO 8 UNI GX 0.04
*
*
*
LOAD 11 THERMAL EXPANSION FORCE (T)
*DELTA T OF 0 DEGREES APPLIED TO ALL STRUCTURAL MEMBERS
TEMPERATURE LOAD
LOAD 15 NOTIONAL LOAD (N)
*LOAD 1 AND 2 MULTIPLIED BY 0.0323 AND APPLIED LATERALLY
*BASED ON 1/DELTA = 65
*LOAD 1 DEAD LOAD (DS) STEEL AND FIREPROOFING
*SELFWEIGHT OF THE STRUCTURE PLUS 10% FOR CONNECTIONS
  SELFWEIGHT X 0.03553
*
*LOAD 2 OPERATING DEAD LOAD (DO)
*40 PSF UNIFORM LOAD OVER TRIBUTARY AREA
*40PSF*20FEET = 800 PLF = 0.8 KLF
MEMBER LOAD
  5 TO 8 UNI GX 0.02584
*

LOAD 20 NATURAL PERIOD CALCULATION
*LOAD 1 DEAD LOAD (DS) STEEL AND FIREPROOFING
  SELFWEIGHT x -1.1
*LOAD 2 OPERATING DEAD LOAD (DO)
MEMBER LOAD
  5 TO 8 UNI Gx -0.8
*LOAD 3 EMPTY DEAD LOAD (DE)
MEMBER LOAD
  5 TO 8 UNI Gx -0.48
  modal calculation requested
*

*********************************************************************************
* ASD LOAD COMBINATIONS
*********************************************************************************

* DS+DO+FF+T+AF
*********************************************************************************
LOAD COMB 101 DS+DO+FF+T+AF
  1 1.0 2 1.0 9 1.0 11 1.0 10 1.0
*
LOAD COMB 102 DS+DO-FF+T-AF
  1 1.0 2 1.0 9 -1.0 11 1.0 10 -1.0
*
LOAD COMB 103 DS+DO+FF-T+AF
  1 1.0 2 1.0 9 1.0 11 -1.0 10 1.0
*
LOAD COMB 104 DS+DO-FF-T-AF
  1 1.0 2 1.0 9 -1.0 11 -1.0 10 -1.0
*
*********************************************************************************
* DS+DO+AF+W
*********************************************************************************
LOAD COMB 105 DS+DO+AF+WX
1 1.0 2 1.0 10 1.0 5 1.0
*
LOAD COMB 106 DS+DO-AF+WX
1 1.0 2 1.0 10 -1.0 5 1.0
*
LOAD COMB 107 DS+DO+AF-WX
1 1.0 2 1.0 10 1.0 5 -1.0
*
LOAD COMB 108 DS+DO-AF-WX
1 1.0 2 1.0 10 -1.0 5 -1.0
*
**************************************************
* DS+DO+AF+0.7E
**************************************************
LOAD COMB 109 DS+DO+AF+0.7EX+0.7EY
1 1.0 2 1.0 10 1.0 7 0.7 14 0.7
*
LOAD COMB 110 DS+DO-AF+0.7EX+0.7EY
1 1.0 2 1.0 10 -1.0 7 0.7 14 0.7
*
LOAD COMB 111 DS+DO+AF-0.7EX+0.7EY
1 1.0 2 1.0 10 1.0 7 -0.7 14 0.7
*
LOAD COMB 112 DS+DO-AF-0.7EX+0.7EY
1 1.0 2 1.0 10 -1.0 7 -0.7 14 0.7
*
**************************************************
* DS+DE+W
**************************************************
LOAD COMB 113 DS+DE+WX
1 1.0 3 1.0 5 1.0
*
LOAD COMB 114 DS+DE-WX
1 1.0 3 1.0 5 -1.0
*
**************************************************
* 0.9DS+0.6DO+AF+0.7E
**************************************************
LOAD COMB 115 0.9DS+0.6DO+AF+0.7EX-0.7EY
1 0.9 2 0.6 10 1.0 7 0.7 14 -0.7
*
LOAD COMB 116 0.9DS+0.6DO-AF+0.7EX-0.7EY
1 0.9 2 0.6 10 -1.0 7 0.7 14 -0.7
*
LOAD COMB 117 0.9DS+0.6DO+AF-0.7EX-0.7EY
1 0.9 2 0.6 10 1.0 7 -0.7 14 -0.7
*
LOAD COMB 118 0.9DS+0.6DO-AF-0.7EX-0.7EY
1 0.9 2 0.6 10 -1.0 7 -0.7 14 -0.7
* 0.9(DS+DE)+0.7EE
* LOAD COMB 119 0.9(DS+DE)+0.42EX-0.42EY
 1 0.9 3 0.9 7 0.42 14 -0.42
* LOAD COMB 120 0.9(DS+DE)-0.42EX-0.42EY
 1 0.9 3 0.9 7 -0.42 14 -0.42
* DS+DT+WP
* LOAD COMB 121 DS+DT+WPX
 1 1.0 4 1.0 5 0.56
* LOAD COMB 122 DS+DT-WPX
 1 1.0 4 1.0 5 -0.56
* LRFD LOAD COMBINATIONS (WITH NOTIONAL)
* 1.4(DS+DO+FF+T+AF)
* LOAD COMB 201 1.4(DS+DO+FF+T+AF+NX)
 1 1.4 2 1.4 9 1.4 11 1.4 10 1.4 15 1.4
* LOAD COMB 202 1.4(DS+DO-FF+T-AF-NX)
 1 1.4 2 1.4 9 -1.4 11 1.4 10 -1.4 15 -1.4
* LOAD COMB 203 1.4(DS+DO+FF-T+AF+NX)
 1 1.4 2 1.4 9 1.4 11 -1.4 10 1.4 15 1.4
* LOAD COMB 204 1.4(DS+DO-FF-T-AF-NX)
 1 1.4 2 1.4 9 -1.4 11 -1.4 10 -1.4 15 -1.4
* 1.2(DS+DO+AF)+1.6W
* LOAD COMB 205 1.2(DS+DO+AF+NX)+1.6WX
 1 1.2 2 1.2 10 1.2 5 1.6 15 1.2
* LOAD COMB 206 1.2(DS+DO-AF+NX)+1.6WX
 1 1.2 2 1.2 10 -1.2 5 1.6 15 1.2
* LOAD COMB 207 1.2(DS+DO+AF-NX)-1.6WX
 1 1.2 2 1.2 10 1.2 5 -1.6 15 -1.2
*
LOAD COMB 208 1.2(DS+DO-AF-NX)-1.6WX
1 1.2 2 1.2 10 -1.2 5 -1.6 15 -1.2
*
LOAD COMB 209 1.2(DS+DO+AF-NX)+1.6WX
1 1.2 2 1.2 10 1.2 5 1.6 15 -1.2
*
LOAD COMB 210 1.2(DS+DO-AF-NX)+1.6WX
1 1.2 2 1.2 10 -1.2 5 1.6 15 1.2
*
LOAD COMB 211 1.2(DS+DO+AF+NX)-1.6WX
1 1.2 2 1.2 10 1.2 5 -1.6 15 1.2
*
LOAD COMB 212 1.2(DS+DO-AF+NX)-1.6WX
1 1.2 2 1.2 10 -1.2 5 -1.6 15 1.2
*
**************************************************
* 1.2(DS+DO+AF)+1.0Eo
**************************************************
LOAD COMB 213 1.2(DS+DO+AF+NX)+1.0EX+1.0EY
1 1.2 2 1.2 10 1.2 7 1 14 1.0 15 1.2
*
LOAD COMB 214 1.2(DS+DO-AF+NX)+1.0EX+1.0EY
1 1.2 2 1.2 10 -1.2 7 1 14 1.0 15 1.2
*
LOAD COMB 215 1.2(DS+DO+AF-NX)-1.0EX+1.0EY
1 1.2 2 1.2 10 1.2 7 -1.0 14 1.0 15 1.2
*
LOAD COMB 216 1.2(DS+DO-AF-NX)-1.0EX+1.0EY
1 1.2 2 1.2 10 -1.2 7 -1.0 14 1.0 15 1.2
*
LOAD COMB 217 1.2(DS+DO+AF-NX)+1.0EX+1.0EY
1 1.2 2 1.2 10 1.2 7 1 14 1.0 15 -1.2
*
LOAD COMB 218 1.2(DS+DO-AF-NX)+1.0EX+1.0EY
1 1.2 2 1.2 10 -1.2 7 1 14 1.0 15 -1.2
*
LOAD COMB 219 1.2(DS+DO-AF-NX)-1.0EX+1.0EY
1 1.2 2 1.2 10 1.2 7 -1.0 14 1.0 15 -1.2
*
LOAD COMB 220 1.2(DS+DO-AF-NX)-1.0EX+1.0EY
1 1.2 2 1.2 10 -1.2 7 -1.0 14 1.0 15 -1.2
*
**************************************************
* 0.9(DE+DE)+1.6W
**************************************************
LOAD COMB 221 0.9(DS+DE+NX)+1.6WX
1 0.9 3 0.9 5 1.6 15 0.9
*
LOAD COMB 222 0.9(DS+DE-NX)-1.6WX
1 0.9 3 0.9 5 -1.6 15 -0.9
* 0.9(DS+DO)+1.2AF+1.0E
*******************************************************************************
LOAD COMB 223 0.9(DS+DO+NX)+1.2AF+EX-EY
 1 0.9 2 0.9 10 1.2 7 1.0 14 -1.0 15 0.9
*
LOAD COMB 224 0.9(DS+DO+NX)-1.2AF+EX-EY
 1 0.9 2 0.9 10 -1.2 7 1.0 14 -1.0 15 0.9
*
LOAD COMB 225 0.9(DS+DO+NX)+1.2AF-EX-EY
 1 0.9 2 0.9 10 1.2 7 -1.0 14 -1.0 15 0.9
*
LOAD COMB 226 0.9(DS+DO+NX)-1.2AF-EX-EY
 1 0.9 2 0.9 10 -1.2 7 -1.0 14 -1.0 15 0.9
*
LOAD COMB 227 0.9(DS+DO-NX)+1.2AF+EX-EY
 1 0.9 2 0.9 10 1.2 7 1.0 14 -1.0 15 -0.9
*
LOAD COMB 228 0.9(DS+DO-NX)-1.2AF+EX-EY
 1 0.9 2 0.9 10 -1.2 7 1.0 14 -1.0 15 -0.9
*
LOAD COMB 229 0.9(DS+DO-NX)-1.2AF-EX-EY
 1 0.9 2 0.9 10 -1.2 7 -1.0 14 -1.0 15 -0.9
*
LOAD COMB 230 0.9(DS+DO-NX)-1.2AF-EX-EY
 1 0.9 2 0.9 10 -1.2 7 -1.0 14 -1.0 15 -0.9
*
*******************************************************************************
* 1.4(DS+DT)
*******************************************************************************
LOAD COMB 231 1.4(DS+DT+NX)+EEX-EEY
 1 0.9 3 0.9 7 0.6 14 -0.6 15 0.9
*
LOAD COMB 232 1.4(DS+DT-NX)-EEX-EEY
 1 0.9 3 0.9 7 -0.6 14 -0.6 15 -0.9
*
*******************************************************************************
* 1.4(DS+DT)
*******************************************************************************
LOAD COMB 233 1.4(DS+DT+NX)
 1 1.4 4 1.4 15 1.4
*
LOAD COMB 234 1.4(DS+DT-NX)
 1 1.4 4 1.4 15 1.4
*
*******************************************************************************
* 1.2(DS+DT)+1.6(0.56W)
*******************************************************************************
LOAD COMB 235 1.2(DS+DT+NX)+1.6(0.56WX)
1 1.2 4 1.2 5 0.9 15 1.2
*
LOAD COMB 236 1.2(DS+DT-NX)-1.6(0.56WX)
1 1.2 4 1.2 5 -0.9 15 -1.2
*
*
*
PERFORM ANALYSIS
LOAD LIST 201 TO 236
PARAMETER 1
CODE AISC UNIFIED
FYLD 7200 ALL
*
KZ 1 ALL
KY 1 ALL
*
CHECK CODE ALL
*
FINISH