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# Is Your Non-Building Structure Suitably Braced: A Case Study

Perry S. Green[1] and Kevin Jedrzejczak[2]

**Abstract**
The structural analysis methodology and stability of a 607 ft. (185m) high steel Flare Derrick Structure with base dimensions 105 ft. x 105 ft. (32m x 32m) was reviewed for design adequacy in accordance with the latest 2010 AISC Specification. One of the areas of concern was that the secondary bracing system employed to restrain the primary vertical structural bracing was not triangulated (i.e., secondary bracing was free to move laterally with the primary bracing) but the primary bracing had been designed as though the secondary bracing was an effective restraint point. These concerns were highlighted with an explanation of effective restraint of columns/bracing and the need for K-factor or member effective length parameters in STAAD Pro for cases where bracings have nodes/members attached which do not provide restraint. Both the AISC *Direct Analysis Method* (DAM) and *Effective Length* method (K-factor method) were used and compared. It was noted with the K-factor method that the secondary bracing will share brace reserve strength with neighboring braces and will therefore reduce interaction ratios while using the DAM will only produce correct results when the columns and bracings are directly impacted by the notional loads and P-Δ forces (primary bracing only columns only) which is not the case for this structure. In order to utilize the *Direct Analysis Method* correctly notional offsets were employed; their development and STAAD Pro results are described in this paper.

## 1. Introduction

The structural design, analysis and subsequent construction of a 607 ft. (185m) high Flare Derrick Structure with base dimensions 105 ft. x 105 ft. (32m x 32m) was completed in 2014 (see Fig. 1 for an overall graphical representation of the tower). A sanity check of the overall tower's structural design was undertaken including a detailed review of the tower's structural models that had been created in STAAD Pro (2010). One of the areas of concern that arose during the review was that the secondary bracing system used to restrain the primary vertical structural bracing was not triangulated (i.e., secondary bracing was free to move laterally with the primary bracing) but the primary bracing had been designed as though the secondary bracing was an effective restraint point. The tower had been analyzed using the AISC360-10 *Effective Length* method (AISC, 2010) since it was known that the effective restraint of the primary columns/vertical bracing, i.e., K-factors greater than unity, was not always provided where the

[1] Senior Civil Engineer, Bechtel NS & E, <pgreen2@bechtel.com>
[2] EGS Civil/Structural Engineer, Bechtel OG &C <kxjedrze@bechtel.com>

horizontal bracings have nodes/members attached which do not provide adequate restraint. In addition to the analyses that were done utilizing the K-factor method, a parallel set of analyses were completed in STAAD Pro based on the AISC360-10 *Direct Analysis Method* to confirm the results of the structural design. In this way there would be no concern as to whether the correct K-factors were employed for all the compression members.

An independent structural review of the Flare Derrick Structure was then carried out in our Frederick, MD office. The review consisted of the following activities:

- **Drawings – design/analysis vs as-built configuration**
    - Member sizes, geometry, properties, types
    - Material properties
- **STAAD Pro model inputs**
    - Member sizes, geometry, properties
    - Material properties
    - Member releases, restraints
    - Load cases and load combinations as per Table 1 (e.g. live loads, wind loads)
    - Solution methodologies - Effective Length method, Direct Analysis Method
- **STAAD Pro model outputs**

The main focus was to confirm the adequacy of the structural design; to verify that the use of DAM was proper considering that its intended use is for columns and bracings which are directly impacted by the notional loads that are determined and applied to the structure that will cause additional, secondary or P-Δ forces (primary bracing only columns only) acting on the structure in a destabilizing condition.

Table 1. Load Cases and Load Combinations Definitions

| | | Load Case | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Load Combination | 1 | 2 | 3 | 6 | 7 | 9 | 10 | 11 | 12 | 39 | 40 | 41 | 42 | 43 | 44 |
| | | DL Frame | DL Platforms | DL Pipes | LL Platforms | LL Pipes | WL X-direction on frame | WL X-direction on risers | WL Z-direction on frame | WL Z-direction on risers | 24" Riser Tilt | 72" Riser Tilt | 66" Riser Tilt | 24" Riser Tilt | 72" Riser Tilt | 66" Riser Tilt |
| Lifting | 100 | 1 | 1 | 1 | 1 | | | | | | 1 | | | | | |
| | 101 | 1 | 1 | 1 | 1 | | | | | | 1 | | | | | |
| | 102 | 1 | 1 | 1 | 1 | | | | | | 1 | | | | | |
| | 103 | 1 | 1 | 1 | 1 | | | | | | 1 | | | | | |
| | 104 | 1 | 1 | 1 | 1 | | | | | | | 1 | | | | |
| | 105 | 1 | 1 | 1 | 1 | | | | | | | 1 | | | | |
| | 106 | 1 | 1 | 1 | 1 | | | | | | | 1 | | | | |
| | 107 | 1 | 1 | 1 | 1 | | | | | | | 1 | | | | |
| | 108 | 1 | 1 | 1 | 1 | | | | | | | | 1 | | | |
| | 109 | 1 | 1 | 1 | 1 | | | | | | | | 1 | | | |
| | 110 | 1 | 1 | 1 | 1 | | | | | | | | 1 | | | |
| | 111 | 1 | 1 | 1 | 1 | | | | | | | | 1 | | | |
| | 112 | 1 | 1 | 1 | 1 | | | | | | | | | 1 | | |
| | 113 | 1 | 1 | 1 | 1 | | | | | | | | | 1 | | |
| | 114 | 1 | 1 | 1 | 1 | | | | | | | | | 1 | | |
| | 115 | 1 | 1 | 1 | 1 | | | | | | | | | 1 | | |
| | 116 | 1 | 1 | 1 | 1 | | | | | | | | | | 1 | |
| | 117 | 1 | 1 | 1 | 1 | | | | | | | | | | 1 | |
| | 118 | 1 | 1 | 1 | 1 | | | | | | | | | | 1 | |
| | 119 | 1 | 1 | 1 | 1 | | | | | | | | | | 1 | |
| | 120 | 1 | 1 | 1 | 1 | | | | | | | | | | | 1 |
| | 121 | 1 | 1 | 1 | 1 | | | | | | | | | | | 1 |
| | 122 | 1 | 1 | 1 | 1 | | | | | | | | | | | 1 |
| | 123 | 1 | 1 | 1 | 1 | | | | | | | | | | | 1 |
| DL+0.6WL | 200 | 1 | 1 | 1 | | 1 | 0.6 | 0.6 | | | | | | | | |
| | 201 | 1 | 1 | 1 | | 1 | -0.6 | -0.6 | | | | | | | | |
| | 202 | 1 | 1 | 1 | | 1 | | | 0.6 | 0.6 | | | | | | |
| | 203 | 1 | 1 | 1 | | 1 | | | -0.6 | -0.6 | | | | | | |
| | 204 | 1 | 1 | 1 | | 1 | 0.471 | 0.425 | 0.471 | 0.425 | | | | | | |
| | 205 | 1 | 1 | 1 | | 1 | -0.471 | -0.425 | -0.471 | -0.425 | | | | | | |
| | 206 | 1 | 1 | 1 | | 1 | 0.471 | 0.425 | -0.471 | -0.425 | | | | | | |
| | 207 | 1 | 1 | 1 | | 1 | -0.471 | -0.425 | 0.471 | 0.425 | | | | | | |
| DL+0.75LL+0.75(0.6WL) | 208 | 1 | 1 | 1 | 0.75 | 1 | 0.45 | 0.45 | | | | | | | | |
| | 209 | 1 | 1 | 1 | 0.75 | 1 | -0.45 | -0.45 | | | | | | | | |
| | 210 | 1 | 1 | 1 | 0.75 | 1 | | | 0.45 | 0.45 | | | | | | |
| | 211 | 1 | 1 | 1 | 0.75 | 1 | | | -0.45 | -0.45 | | | | | | |
| | 212 | 1 | 1 | 1 | 0.75 | 1 | 0.35 | 0.32 | 0.35 | 0.32 | | | | | | |
| | 213 | 1 | 1 | 1 | 0.75 | 1 | -0.35 | -0.32 | -0.35 | -0.32 | | | | | | |
| | 214 | 1 | 1 | 1 | 0.75 | 1 | 0.35 | 0.32 | -0.35 | -0.32 | | | | | | |
| | 215 | 1 | 1 | 1 | 0.75 | 1 | -0.35 | -0.32 | 0.35 | 0.32 | | | | | | |

NOTE: Load Combinations 300 to 303 are D+0.7E
Load Combinations 304 and 305 are D+0.75L+0.75(0.7E)
The lifitng load combinations of 100 to 123 have notional loads in four directions. For example, Load Combinations 100-103 has notional loads in the +X, -X, +Z, and -Z directions.
The factors on the wind loads for load combinations 204 to 207 did not include the 0.6 factor.

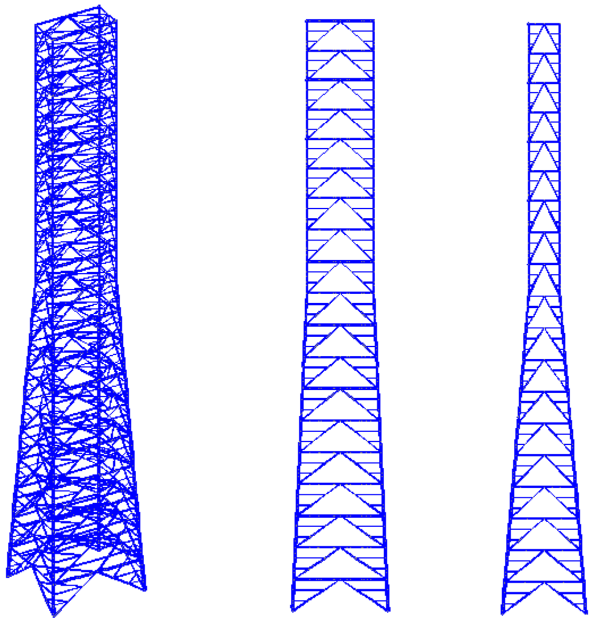


Figure 1. Flare Derrick Structure - Overall View STAAD Pro Model

## 2. Bracing Configurations of Steel Lattice Towers

Free-standing, 4-legged steel lattice towers have been constructed for many different purposes such as radio, television, microwave, or satellite antenna/communication, energy transmission, petrochemical, etc. The bracing systems of these types of towers will vary significantly depending on their height which can range from less than 100 ft. (30m) to over 1000 ft. (300m). A substantial amount of research dedicated to these various bracing designs and optimization of these systems has been carried out, Dias (2007), Efthymiou et al (2009), Jesumi and Rajendran (2013), Xie and Sun (2013), but has primarily been limited to steel lattice towers less than 200 ft. (60m). Fig. 2 shows some of the more typical bracing systems used for these tower heights and are described as: X-B, Single Diagonal, X-X, K and Y bracing from left to right in the figure.

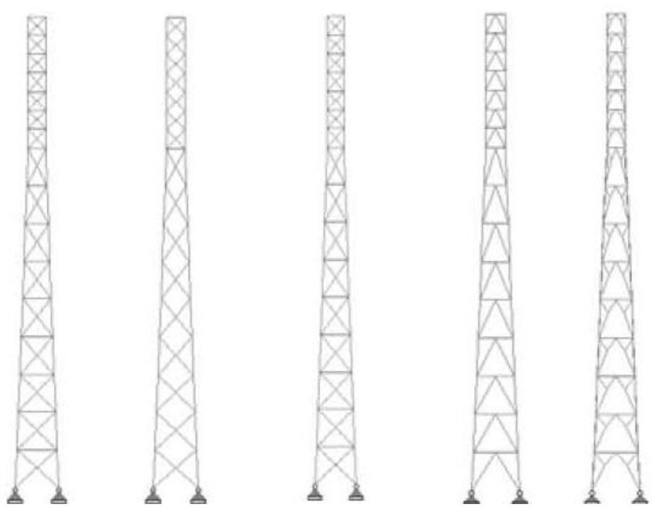


Figure 2. Models of 164 ft. (50m) Towers with different Bracing Systems (Jesumi and Rajendran, 2013)

The treatment of bracing and bracing configurations is explicitly described, though only "informative" in *Eurocode* 3 – Design of steel structures – Part 3-1: Towers, masts and chimneys – Towers and masts Annex H (EN 1993-3-1:2006). Typical primary and secondary bracing patterns are given, but these are based on permissive rules and not mandatory (see Fig. 3).

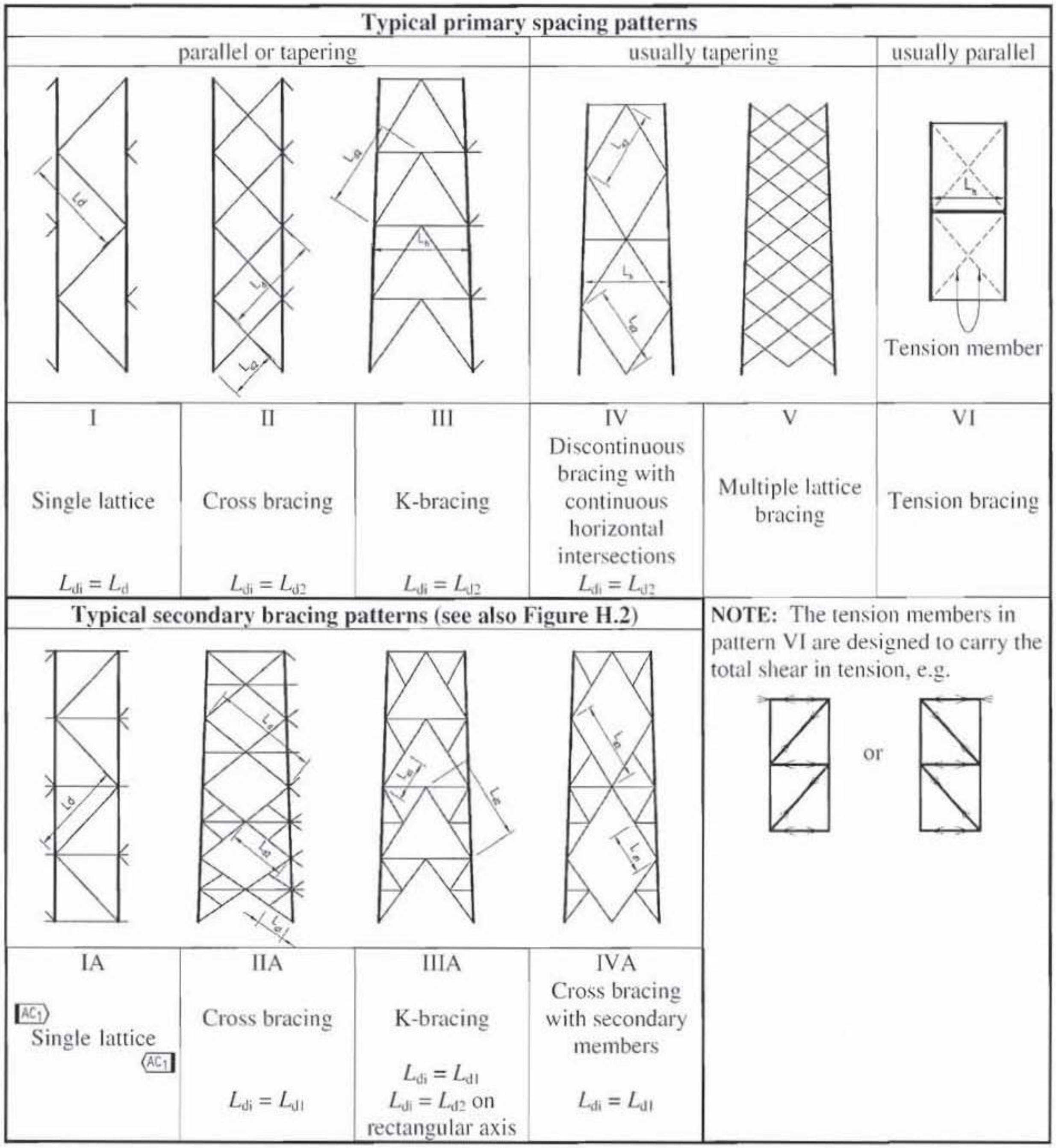



Figure 3. Typical Bracing Patterns (EN 1993-3-1:2006)

Additionally, Annex H provides information regarding plan bracing when the system(s) is either triangulated or not fully triangulated as shown in Fig. 4. This is important as will be shown later as it applies specifically to the steel lattice structure that is being reviewed.

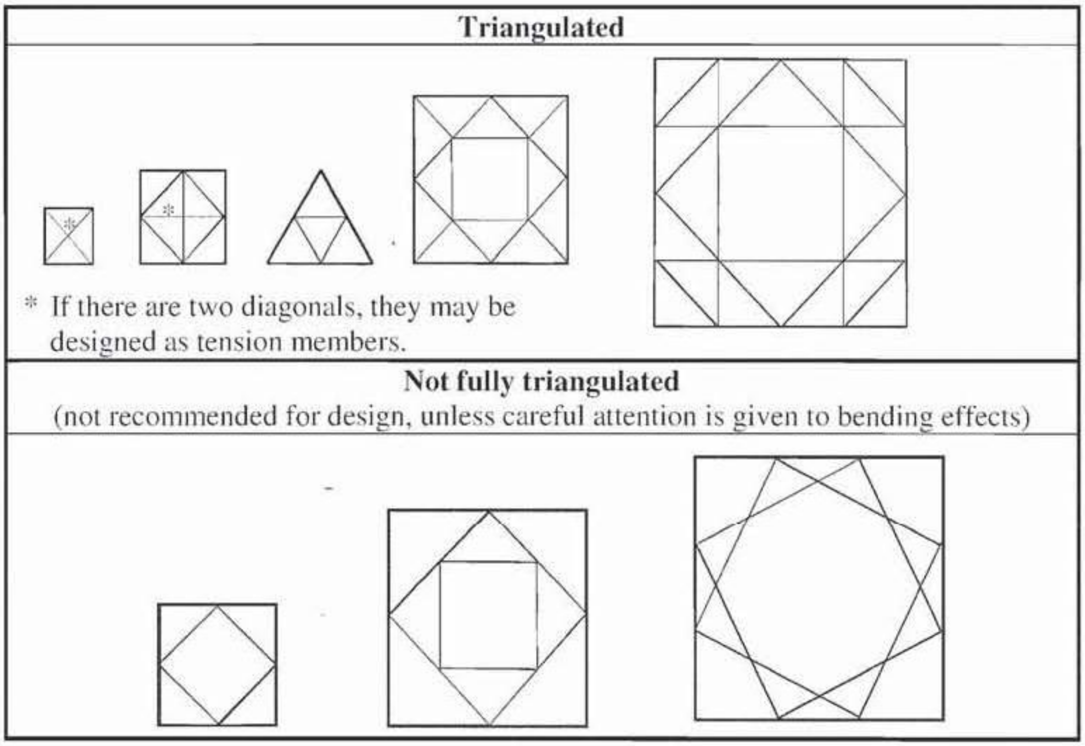


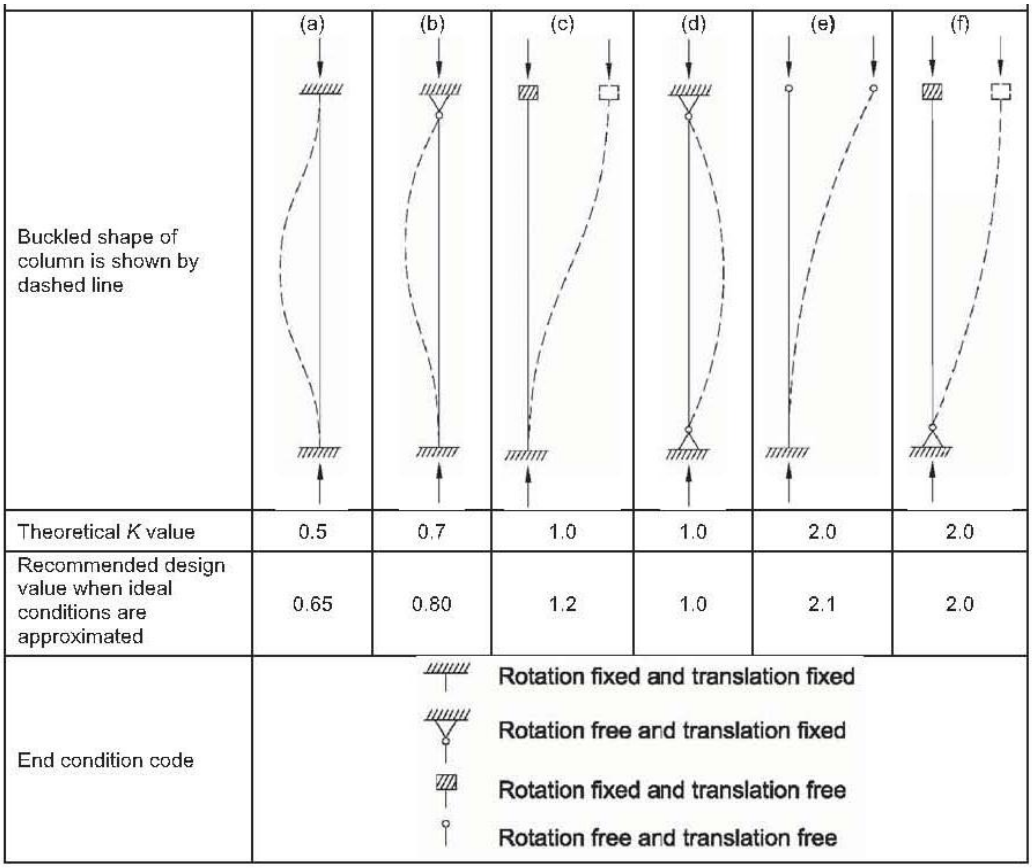

Figure 4. Typical Plan Bracing (EN 1993-3-1:2006)

## 3. Effective Length Method

### *3.1. Bracing effective lengths in trussed structures*

In a trussed structure, as the Flare Derrick Structure, it is understood that the loads would be applied at the joints. If the joints where the bracing members are connected to the primary, compression leg members are truly pinned then the primary members would only be axially loaded. Even if these joints are comprised of a welded gusset plate type connection between the bracing member and the primary member, there may be some restraint that would produce secondary bending stresses, but these would typically have little to no effect on the overall buckling strength of the primary members because local yielding would take place in the most flexible portion of the connection thereby dissipating any secondary moments allowing the primary member to reach its ultimate strength.

When a truss is designed and loaded such that all members reach their factored resistances simultaneously, no member restrains any other. Therefore, the effective-length factors for compression members and compression braces would be 1.0 for in-plane buckling (Ziemian, 2010). A simple bracing arrangement would be one that is supported at its ends without an intermediate restraint. This representation is equivalent to condition (d) in Fig. 5 that reproduces Table C-A-7.1 from the AISC360-10 Commentary *Appendix 7, Alternative Methods of Design for Stability* (AISC, 2010). In designing a proper bracing system the braces need to have sufficient strength as well as stiffness to act as an effective bracing member. In the case that is being described in this paper the primary bracing members, either due to their length and/or cross-section needed to have their own restraint, typically being provided by secondary horizontal or hip bracings. It is this overall bracing system that is being looked at critically as it ultimately had an effect on the overall member design of the Flare Derrick Structure.



| Buckled shape of column is shown by dashed line | (a) | (b) | (c) | (d) | (e) | (f) |
|---|---|---|---|---|---|---|
| Theoretical *K* value | 0.5 | 0.7 | 1.0 | 1.0 | 2.0 | 2.0 |
| Recommended design value when ideal conditions are approximated | 0.65 | 0.80 | 1.2 | 1.0 | 2.1 | 2.0 |
| End condition code | Rotation fixed and translation fixed<br>Rotation free and translation fixed<br>Rotation fixed and translation free<br>Rotation free and translation free | | | | | |

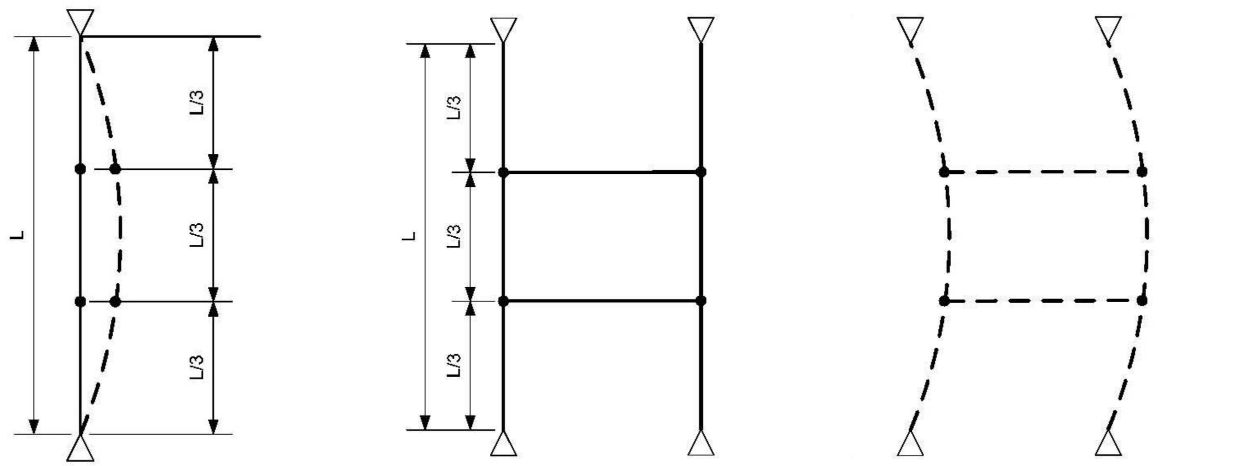
Figure 5. Table C-A-7.1 Approximate Values of Effective Length Factor, *K* (AISC, 2010)

### *3.2. Effective Length Simplified Example*

Given a vertical brace with two intermediate node points as shown in Fig. 6a it would be designed using STAAD Pro based on the following factors:

- The member would be designed for an overall length of “L”,
- The nodes within the member length would make the STAAD Pro length default to L/3,
- The member design parameters in STAAD Pro would be entered manually to override the default length to state either member length is “L”, or K-factor for the default length (not manually specified) of L/3 is “3”. STAAD Pro will then multiply “L/3” by “3” to use length “L” for member design checks for overall column buckling.

Now let us assume that we have two columns as detailed above but they are joined by two bracing members at the node points as shown in Fig. 6b. The columns (representing the primary vertical or chevron bracings) are joined together by the horizontal members (representing the secondary or hip bracings) at the one-third point locations. The members are not “triangulated” and therefore they are free to move with the columns at the points where they are connected to the columns. In this example the columns have the same geometric and material properties and have the same failure load as though they were not braced. They are restrained from failing in opposite directions, but they are free to fail in the same direction as shown in Fig. 6c. If the columns were to fail in this manner their buckling load would be the same as though the columns did not have any members joining them together at their one-third points, i.e., the same as the single column illustrated in Fig. 6a.




a) One Pin-Ended Column After Buckling Failure

b) Two Pin-Ended Columns Braced Together at the One-Third Points, Before Buckling Failure

c) Two Pin-Ended Columns Braced Together at the One-Third Points, After Buckling Failure

Figure 6.Pin-Ended Column Example Showing Overall Buckling of a) Single Pin-Ended Column; b) Two Pin-Ended Columns with c) Ineffective Bracings

## 4. Simplified STAAD Pro Models Used to Demonstrate Problem

A set of STAAD Pro files were prepared which demonstrate that the *Direct Analysis Method* will not give comparable results to the *Effective Length* method for an effective length based on a pin-ended column with K=1. A review of available code references, presentations by AISC, papers etc. all indicate that DAM is intended for primary columns and bracings directly impacted by notional loads and P-Δ forces. All the references apply effective length at restrained location to restrained location with a K-factor equal to 1 (unity). No examples could be found that use ineffective secondary bracing of primary bracing and assume DAM will take this into account for the design of the primary bracing.

Fig. 7 shows the seven simplified models that were created to check the proper usage of DAM in STAAD Pro. The models were simple design problems with secondary bracing or unrestrained nodes. First, a W8X31 column section which is 20 ft. (6m) long, made from 36 ksi steel was created to illustrate that it can safely carry just over 90 kips (400 kN) when designed according to AISC360-10, ASD method. Other sample models were created with different layouts of W8x31 members, all with loads of 90 kips (400 kN) and real effective lengths of 20 ft. (6 m). The models typically have unrestrained internal nodes, or ineffective horizontal members.

For the models shown in Fig. 7 all W8x31 effective lengths have been entered in STAAD Pro as 6 m (except for the explicitly braced bay example shown at bottom right of the models which is 6.7 ft. (2m) for weak axis buckling and 20 ft. (6m) for major axis buckling). Fig. 8 gives the STAAD Pro results for the models. It should be noted that the unity ratio check is typically 0.95 for the 20 ft. (6m) length and 0.6 for 6.7 ft. (2m) length.

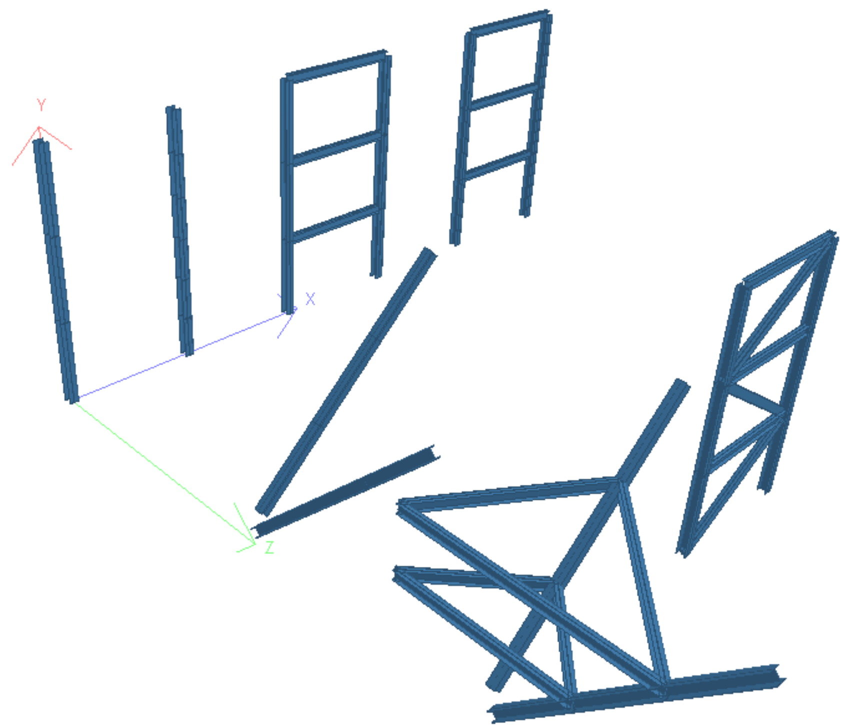



Figure 7. Simplified STAAD Pro Models to Demonstrate Problem

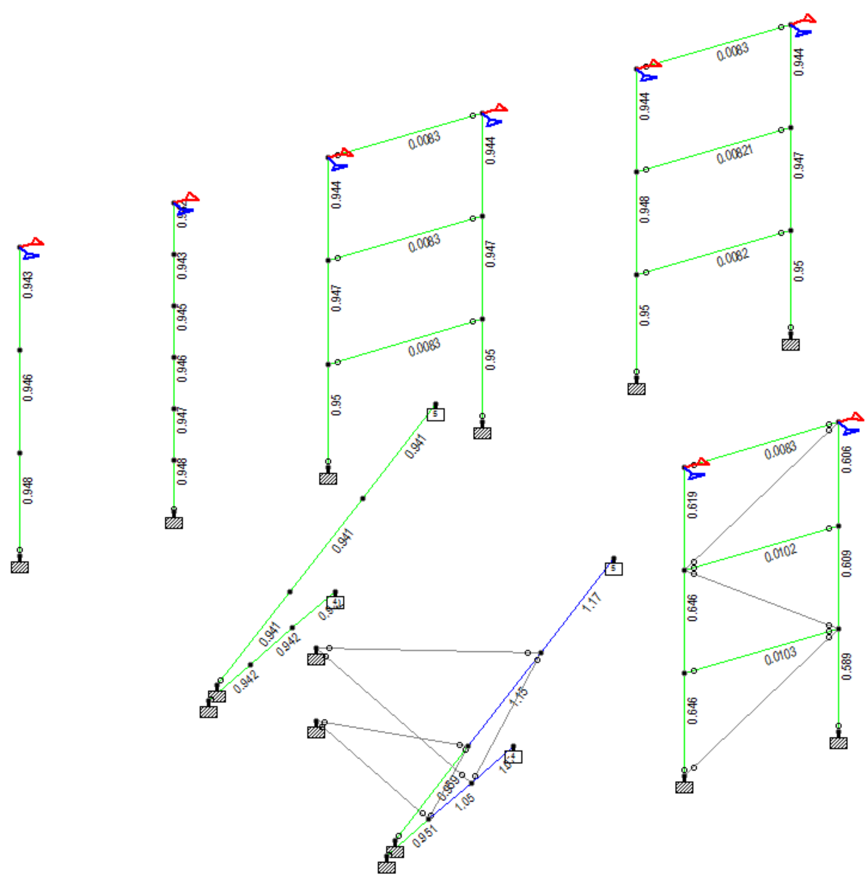



Figure 8. STAAD Pro Effective Length Method Results Showing Unity Ratio Checks

The Direct Analysis Method was then applied to the same STAAD Pro models illustrated in Fig. 7. The member lengths were left to default node-node distances for member design and the notional loads were left to default in STAAD Pro to be applied at locations of load application as part of the DAM analysis. Fig. 9 gives the unity ratio check of 0.5 for a column with one-third the length (L/3) broken by unrestrained nodes or pin-jointed beams acting as ineffective bracing. The unity ratio check for a W8x31 which is 6.7 ft. (2m) long and subject to 90 kips (400 kN) axial load (using Mathcad design) is 0.511.

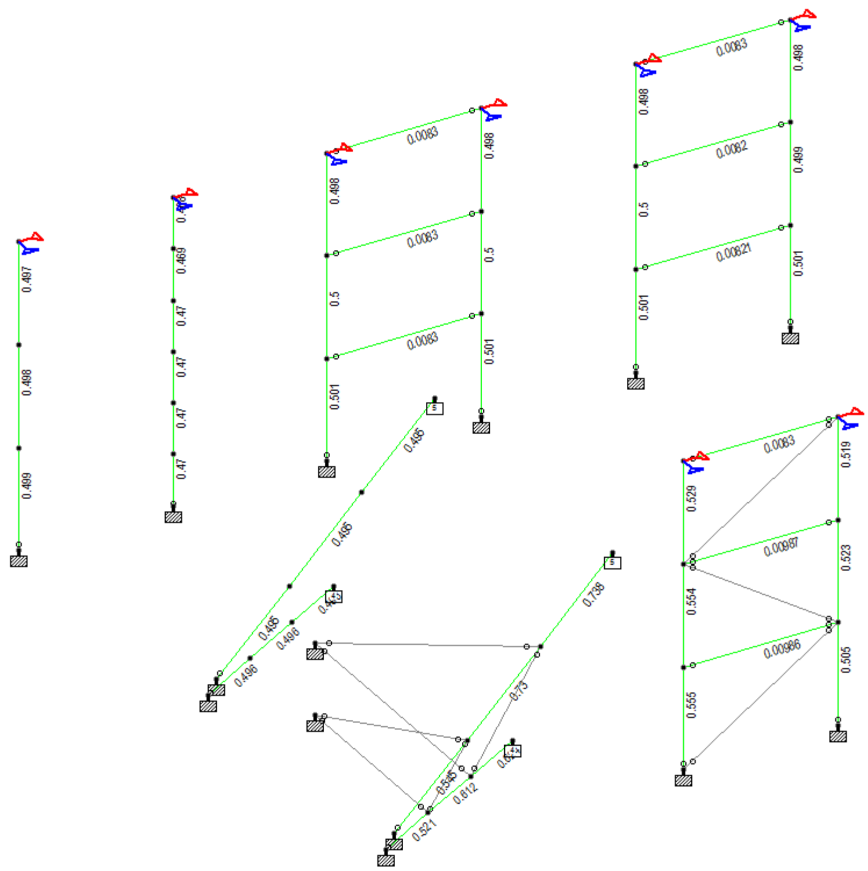


Figure 9. STAAD Pro Direct Analysis Method Results Showing Unity Ratio Checks

The DAM when applied as illustrated by the member designs in Fig. 7 with ineffective bracing, designs those members as though the bracing is effective. As a result of these simplified model examples, it was considered that this analysis approach would be ineffective in the Flare Derrick Structure as the compression member's lateral loading for the design would not be impacted by the notional loading when applied to the tower's structural model.

As a final confirmatory check, notional loading was applied at internal load points in the simplified models. It was found that if DAM was used with a notional load of 0.004 and this was equally and laterally distributed over the internal nodes of the column/brace the resulting unity ratio check for a column with an effective length equal to the node-node distance would be similar to a traditional design for a column with an effective length equal to the column end-end distance. The STAAD Pro results are shown in Fig. 10.

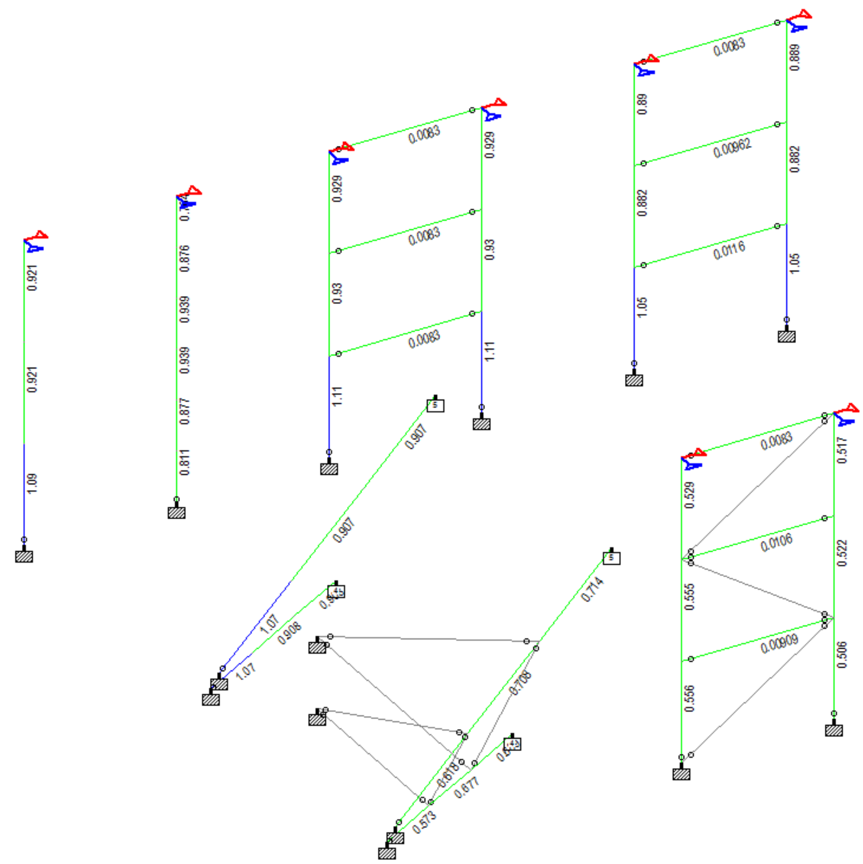


Figure 10. STAAD Pro Direct Analysis Method Results Showing Unity Ratio Checks

## 5. Modified Direct Analysis Method – Notional Displacements

Fig. 11 shows the basic bracing arrangement on the Flare Derrick Structure where each face has two vertical or chevron braces and these vertical braces are themselves braced horizontally in what appears to be a diamond pattern at approximately their one-third points. As this is truly a 3D structural layout, it will be demonstrated that so long as there is no triangulation in more than one plane and there is a direction in which the vertical brace(s) can fail which does not cause the adjoining members, i.e., the horizontal or hip bracings, to shorten (compression)/elongate (tension) then the joining member is not acting as a restraint and the vertical bracing should be designed using an ideal effective length K=3. Figs 12 and 13 schematically show a cross section of the tower in plan as well as an isometric of a representative tier, respectively before and after a failure of the vertical bracings to one of the tower's legs occurs. For clarity the structure's main support legs and plan members are shown with just a centerline pattern and light line weight. The primary and secondary bracing before failure are shown solid. The chevron braces which are shown in the example of failing are shown heavy and solid before failure and dotted and heavy after failure.

As can be seen in Figs. 12 and 13 the vertical bracing can fail in a manner in which the secondary hip bracing member(s) can follow similar to a rigid body rotation. What is meant by this is the secondary bracing member(s) is not restraining the primary vertical brace from failing in that particular mode. The secondary bracing member(s) is ineffective and therefore the primary bracing member(s) must be analyzed in STAAD Pro using an ideal K-factor of 3 when engaging the Effective Length method. More refined K-factors can be specified once the member restraint provided can be determined. Since a significant number of the joints occur between the primary chevron bracing and the tower's column legs or the main tier floor framing a typical K value for these members was taken as 0.95*0.95*3 or 2.7 based on a reduction in the actual brace length considering the joint configuration as described in Fig. 14.

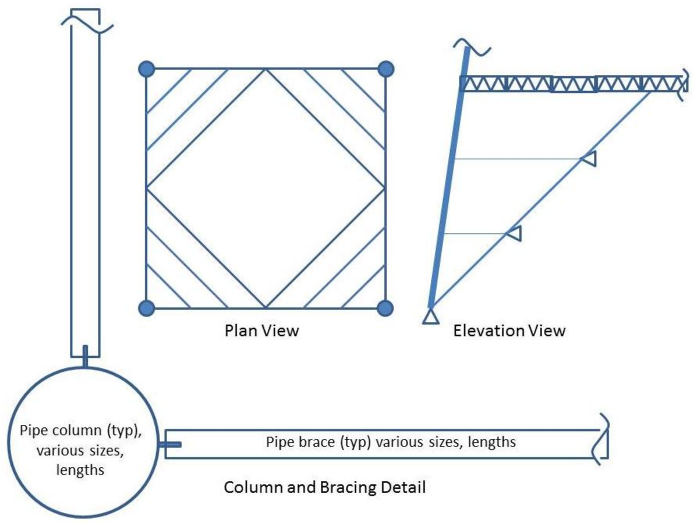
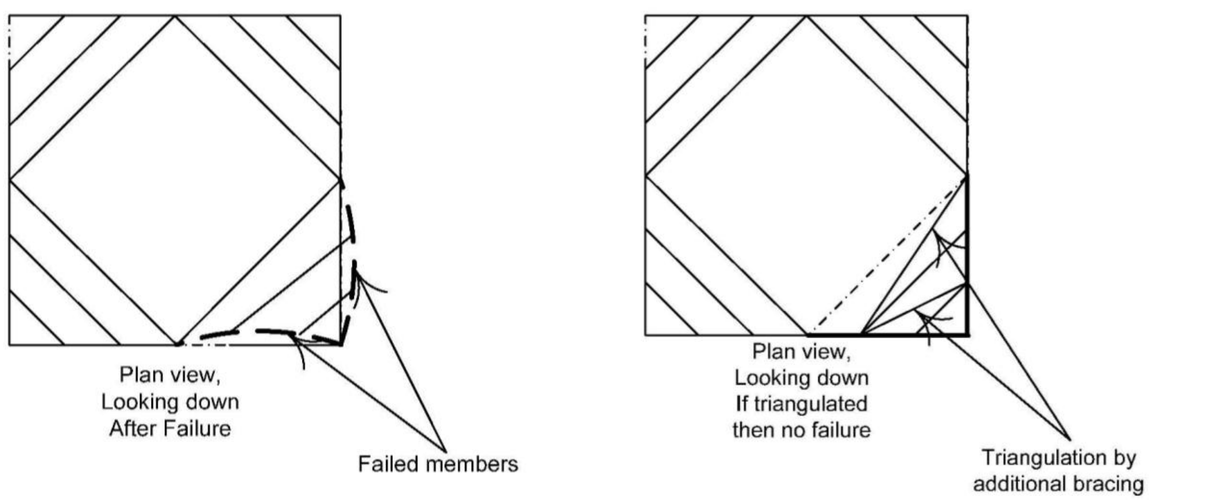


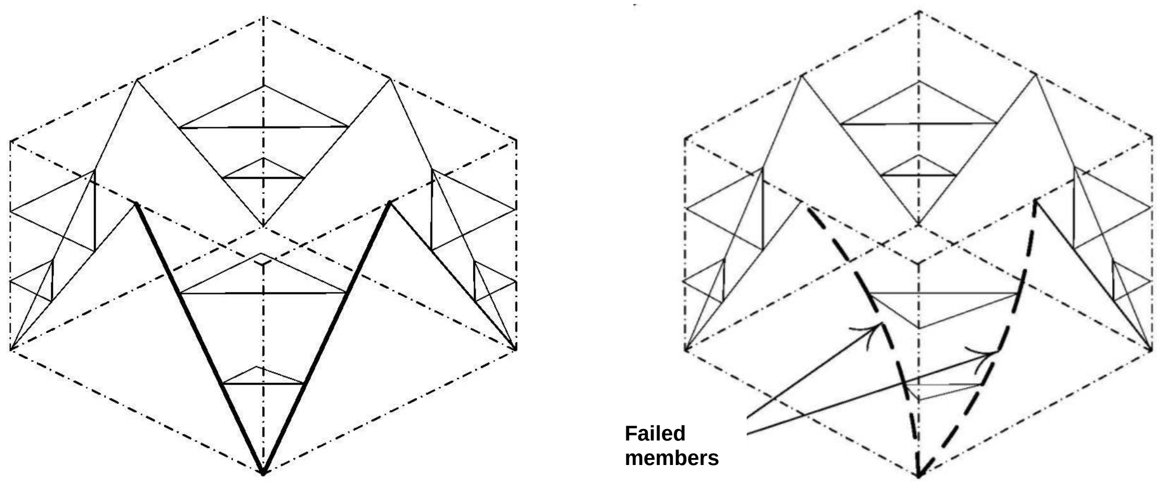

Figure 11. General Column and Bracing Detail, Plan and Elevation Views




a. General Plan View with Current Bracing Scheme    b. General Plan View with Modified Bracing Scheme

Figure 12. General Plan View of Vertical and Horizontal Bracing, a) Showing One Leg with Buckled Vertical Braces, and b) Proposed Bracing Members Added to Prevent Buckling Mode as Illustrated in a).

Depicted in Fig. 13 is a potential modification where two new bracing members have been added to the existing bracing scheme that would prevent failure of the primary vertical bracing by triangulation out-of-plane of the existing secondary horizontal bracing. This could be done by adding members as shown in the figure (these would need to be field welded). Further analytical studies would need to be carried out to determine which vertical bracings and their tier locations must be triangulated to prevent overall buckling failures of primary bracing members. The STAAD Pro model results would need to be reviewed and wherever unity ratio checks exceed 1 at any given tier, strengthening would likely be needed for all secondary bracing members at that elevation.




a. Before Failure

b. After Failure

Figure 13. General Isometric View, a) Typical Vertical and Horizontal Column Bracing Configuration, b) Showing Buckled Vertical Braces Unrestrained by the Horizontal Bracings

| | Type | Axis | AC1 $k^{(3)(5)}$ AC1 |
|---|---|---|---|
| tube or rod legs | single bolted tube | in plane | 0,95[(2)] |
| | | out of plane | 0,95[(2)] |
| | double bolted tube; welded tubes with end plates | in plane | 0,85 |
| | | out of plane | 0,95[(2)] |
| | welded tubes[(1)] and rods with welded gussets | in plane | 0,70 |
| | | out of plane | 0,85 |
| | directly welded tubes and rods | in plane | 0,70 |
| | | out of plane | 0,70 |
| | welded bent rods | in plane | 0,85 |
| | | out of plane | 0,85 |

**NOTE 1:** Double preloaded bolts may qualify for this condition subject to analysis.
**NOTE 2:** Reduction for actual length only, but not less than the distance between end bolts.
**NOTE 3:** Where ends are not the same, an average AC1 "k" AC1 value should be used.
**NOTE 4:** Above details are shown for illustrative purposes only and may not reflect practical design aspects.
**NOTE 5:** Above values are for bracing members with the same connection type at each end. For members with intermediate secondary bracing AC1 "k" AC1 factors may increase and upper values of 1,0 should be used unless justified by tests.

Figure 14. Effective Slenderness Factor K for Bracing Members – Tubes and Rods (EN 1993-3-1:2006)

As a result of this study, the structural models that were developed in STAAD Pro for implementation of the Direct Analysis Method were modified to include notional displacements applied to the vertical braces of the Flare Derrick Structure. These notional (horizontal) displacements were calculated based on an out-of-plane movement in the brace equal to 0.002*L where L is the length of the vertical brace as graphically shown in Figs. 12 and 13. An additional notional load level of 0.0048*Gravity Load in ASD was also required for the DAM.

## 6. Conclusion

The Direct Analysis Method is not intended to be used as a "solve all" solution which removes any need to input the effective length of the bracing nor will it deduce if a bracing system is effective or ineffective and design the structure accordingly. DAM is suitable for the use as described in the ANSI/AISC 360-10 *Specification for Structural Steel Buildings* where it is used in place of calculating items such K-factors by the AISC alignment charts. Properly determined member effective lengths must be entered into the STAAD Pro models and for the complex internal member system employed in the Flare Derrick Structure effective lengths must be reviewed based on an understanding of how STAAD Pro will default to node-node distance and actual points of restraint. When utilizing the Direct Analysis Method in STAAD Pro, load paths of primary and notional loading must be clearly understood to properly apply this analysis method.


## Acknowledgments

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