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Is your non-building structure suitably braced: a third case study in a series

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Abstract

It is incumbent on the designer and/or contractor to ensure all specifications issued to third-party vendors for supplemental designs, including detailing and fabrication, are properly reviewed at the time of issuance and that any calculations prepared and stamped along with the shop drawings be checked and signed off for compliance with all required documents. Several temporary work platform steel structures were needed for this complex project. They were designed, detailed, fabricated, and delivered to the construction project site whereupon they were erected according to the erection drawings provided. Subsequent to the ongoing work in the field to erect these platform structures, it was discovered that several requirements in the performance specification that was issued were not adhered to during the normal process of review, receipt, and approval. When these usual steps are missed, sidestepped, or overlooked there can be severe consequences leading to a partial or full structural collapse as will be shown analytically in the paper. In addition to one or more of the temporary work platforms possibly being compromised, any construction workers using these structures may have been in jeopardy depending on the loading condition(s) placed on the deck of the platforms. It should be noted that all the temporary work platform structures erected will only exist during the construction phase of the project. Once they are no longer needed to move structural assemblies, components, grid and rack modules, and process equipment into the building, they will be disassembled and removed from the site.

1. Introduction

There are instances during complex construction projects when temporary work platforms are needed to assist in the timely installation of supplemental structural steel, electrical, instrumentation, and mechanical equipment, etc., at various floor elevations once there is no other means of getting heavy steel assemblies or modules into the building while the primary structural steel and roof is still under construction. These temporary platform steel structures and their foundations shall be designed to the latest ACI, AISC, ASTM, AWS, RCSC and comply

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with all OSHA Standards. Additional project requirements are unique to the facility being erected and in this case, each temporary work platform was required to meet the following minimum criteria:

- Be designed to support a 30-ton total live load and a 5 kip concentrated load;
- Have removable guardrail/handrails;
- Have safety tie-off points provided in the deck around the perimeter of the work platform (minimum 5000 lb capacity to satisfy OSHA);
- Have a working deck with a flat surface with a skid resistant coating; and
- Utilize bolted connections as much as possible to minimize field welding.

For these installations it was determined that there would be no seismic requirements imposed on the platforms. They were to be designed as freestanding structures with specific lateral bracing elevated, to provide for minimum headroom clearance under the platforms, and with columns located in the corners of the platforms, to allow vehicles/equipment to pass under the platforms.

The temporary work platforms that were evaluated each had a deck that ranged from 1125 to 1350 sq. ft. that was located at the second or third floor elevations of the primary structure, i.e., at 28 ft. and 54 ft. The structural system chosen by the subcontractor was a chevron braced frame in one direction and an x-braced frame in the other direction with floor to floor heights from bottom to top of 14 ft. and 14 ft. for the two-story platforms and 12 ft., 14 ft., 14 ft., and 14 ft. for the three-story platforms. In almost all cases the bracing members were composed of double-angle members. Fig. 1.1a shows an overall view of one of the 54 ft. tall temporary construction work platforms after erection was complete.

The issue that arose during the erection of these temporary work platforms was, simply, that the intermediate connectors that should have been designed, detailed, fabricated, and installed prior to or during erection to have the double angle bracing act as a double-angle cross section in accordance with the AISC 360-10 *Specification for Structural Steel Buildings* (AISC 2010) were not provided causing great concern as to the viability of the structures to perform properly without being retrofitted. Subsequent to the identification of the deficiency a decision was made that intermediate connectors were to be fabricated out of bar stock material and field welded in place (see Fig. 1.1b) to allow the full use of the temporary work platforms. A study was then undertaken to look at the behavior of the temporary work platforms if the structural deficiency had not been discovered as well as the platform's behavior to the performance based loading conditions provided in the contract documents to which they were designed. In addition to the stability analysis conducted, the paper will also address the following four fundamental areas as part of the discussion: Understanding reliance on commercially available structural analysis software, its role in design and analysis, how fabricators rely on complete engineering drawings, and how different procurement, and subcontract approaches can impact project communication.

2. Background – Codes and Specifications

2.1 Construction Practice

The temporary construction work platforms needed to be erected quickly without disruption to the ongoing work fronts around the exterior or interior of the building. Therefore, the designer selected a braced frame structure utilizing structural steel that could be easily fabricated, transported, and assembled from wide flange beam and column members with shop welded attachments or gusset plates that would receive angle bracing members to be field-bolted to the plates. All shop welding was in accordance with AWS D1.1 (AWS 2015) and all structural bolting was in accordance with the Research Council on Structural Connections (RCSC 2009).

(a) (b)

Figure 1.1: Typical 54 Foot Temporary Construction Work Platform (a) Overall View; (b) Close-up View of Chevron Bracing After Immediate Connectors Added

2.2 Materials

2.2.1 Foundations

The temporary construction work platform foundations were comprised of a 2 ft. thick reinforced concrete mat with 6 - 3 ft. x 3 ft. piers approximately 8 ft. tall. Specified minimum concrete strength was $f_c' = 4000$ psi. Foundations and piers were backfilled and compacted to provide full axial and lateral support at the base of the structures.

2.2.2 Superstructures

The temporary construction platform superstructures were comprised of structural steel beams and column members fabricated from ASTM A992 (F_y = 50 ksi) material while the angle bracing members, channels, connecting plates, baseplates, and other miscellaneous pieces were fabricated from ASTM A36 (F_y = 36 ksi) material. A grouted baseplate holding 4 - anchor rods spaced 12" on center and having a nominal embedment depth of 2'-6" was placed at the top of each pier. The anchor rods were ASTM F1554 Grade 36 (F_y = 36 ksi) material. All structural bolts were specified as ASTM F3125 Grade A325; all non-structural bolts were ASTM A307.

2.3.3 Connections

Whenever possible shop welded, field-bolted double-angle clips were utilized for beam to beam or beam to girder connections, otherwise field-bolted connections were made to shop welded shear tabs installed on the beams or girders. Beam to column connections were typically shop-bolted, field-bolted double-angle clip connections at locations where there were no bracing attachments, but were a combination of shop-welded, field-bolted and shop-bolted, field-bolted double-angle clip connections where bracing members would be attached to existing shop welded gusset plates. Bracing members used either a 2-bolt or 3-bolt connection to shop welded attachment or gusset plates that were provided with the beams or columns as appropriate.

2.4 Loading

The platform designer made general use of the loads and load combinations from IBC 2012 for the analysis of the temporary construction work platforms.

The commercial software used determined the self-weight of the structure internally. In addition, the platform deck was covered with 3/4" steel plate or an equivalent 30 psf applied over the entire surface. The only other loadings given to the designer were a 30 ton (60 kip) live load and a 5 kip concentrated load acting on the platform deck. From a review of the 3-D model it was determined that a 125 psf live loading was conservatively applied over the working surface of the platform deck. This loading was meant to represent a uniform pressure of up to two 30 kip grid/rack modules side-by-side, each having a footprint ranging from 25-40 ft. long x 10 ft. wide (75-120 psf). Pattern loading that may have provided maximum moments or shears was not considered in the analysis. Additionally, a 50 psf dead load was used.

Finally, the wind load that could have been calculated for an open type structure such as this platform was replaced with a 20% side impact (lateral load) equal to 12 kips acting along the deck of the structure consistent with ASCE7-10 Section 4.9.4. This was considered acceptable since the time duration that any given module was left exposed on the platform would be minimal. Also, no grid/rack module was allowed to be lifted and placed on the platform deck when the locally measured wind speed exceeded 25 mph. This was a project/site safety requirement.

3. Single Angle and Double-Angle Compression Member Behavior

3.1 Single Angles

The history into the development of single angle compression member design provisions in the AISC Specifications has been carefully documented by Lutz (2017). The ANSI/AISC 360-10, Specification for the Design of Steel Buildings (AISC 2010) Chapter E, Design of Members for Compression and more specifically, E5. Single Angle Compression Members govern the design of both equal and unequal leg single angles.

What is known about these types of compression members (see Fig. 3.1) is:

- Sections always rotate about the shear center; the shear center lies on the axis of symmetry; Flexural-torsional buckling will occur by twisting about a point in the plane of symmetry
- Unsymmetric shapes such as single angles with unequal legs, flexural-torsional buckling will normally control
- The effects of eccentricity on single angle members are permitted to be neglected when the members are evaluated as axially loaded compression members using effective slenderness ratios as provided in Section E5, provided that:

- Members are loaded at the ends in compression through the same one leg
  - Members are attached by welding or by minimum two-bolt connections; and
  - No intermediate transverse loads are present

- For singly symmetric sections such as equal leg single angles, which are commonly used as truss members and bracing members, flexural buckling about the x axis (z axis for angles) or flexural-torsional buckling about the y axis (w axis for angles) can occur.

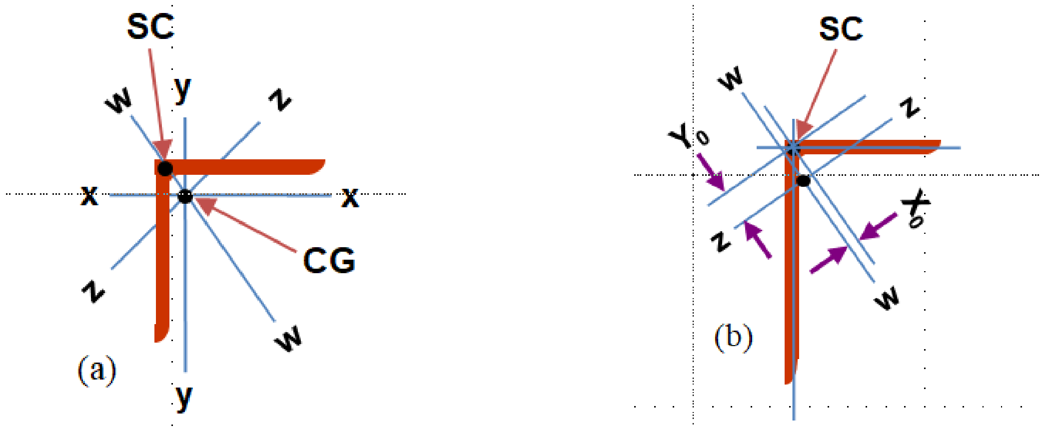


Figure 3.1: Single Angles – a) Equal Leg Properties; and b) Unequal Leg Properties

3.2 Single Angle Bracing Member Design Example

Determine the axial capacity of a L6 x 6 x 5/8 single angle bracing member with an effective length, KL = 10.0 ft.

NOTE: This example only uses Chapter E, Section 4 as the end connection of the single angle is not defined. If more information was provided Section 5 (Equations E5-1 and E5-2) could have been followed which uses a modified slenderness approach.

Material Properties: $F_y = 50$ ksi $\quad$ $E = 29{,}000$ ksi $\quad$ $G = 0.385E$

Section Properties:

$A = 7.13$ in.^{2}	$J = 0.955$ in.^{4}	$r_x = 1.84$ in.
$I_x = 24.1$ in.^{4}	$C_w = 2.5$ in.^{6}	$r_z = 1.17$ in.
$I_y = 24.1$ in.^{4}	$H = 0.631$	$r_o = 3.28$ in.

Determine Flexural Buckling capacity about Z-Z axis,

$$\left(\frac{KL}{r}\right)_z = \frac{(12)(10)}{1.17} = 103 \leq 4.71\sqrt{\frac{E}{F_y}} = 113$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = 23.17 \text{ ksi} \qquad \text{(AISC E3-2)}$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)_z^2} = 27.21 \text{ ksi} \qquad \text{(AISC E3-4)}$$

$$P_n = A_g F_{cr} = 165.20 \text{ kips} \qquad \text{(AISC E3-1, E4-1)}$$

Therefore, ASD $\frac{P_n}{\Omega} = 98.92$ kips LRFD $\phi P_n = 148.68$ kips

Determine Flexural-Torsional buckling capacity

$$I_z = A_g r_z^2 = (7.13)(1.17)^2 = 9.76 \text{ in.}^2 \qquad (3.1)$$

$$I_w = I_x + I_y - I_z = 2(24.1) - 9.76 = 38.44 \text{ in.}^4 \qquad (3.2)$$

$$r_w = \sqrt{\frac{I_w}{A_g}} = \sqrt{\frac{38.44}{7.13}} = 2.32 \text{ in.} \qquad (3.3)$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right) \frac{1}{A_g \bar{r}_o^2} \cong \frac{GJ}{A_g \bar{r}_o^2} \qquad \text{(AISC E4-9)}$$

Note: The first term in the expression is small compared to GJ, therefore this term can be safely ignored for angles, double angles and tees. In this case, 50 versus 10,700.

$$F_{ez} = \frac{GJ}{A_g \bar{r}_o^2} = \frac{(11200)(0.955)}{(7.13)(3.28)^2} = 139.44 \text{ ksi} \qquad (3.4)$$

$$F_{ew} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)_w^2} = 106.98 \text{ ksi} \qquad \text{(AISC E3-4, E4-7, E4-8)}$$

$$F_e = \frac{F_{ew} + F_{ez}}{2H}\left[1 - \sqrt{1 - \frac{4 F_{ew} F_{ez} H}{(F_{ew} + F_{ez})^2}}\right] \qquad \text{(AISC E4-5)}$$

$$F_e = \frac{106.98 + 139.44}{2(0.631)}\left[1 - \sqrt{1 - \frac{4(106.98)(139.44)(0.631)}{(106.98 + 139.44)^2}}\right] = 74.90 \text{ ksi}$$

$$\left(\frac{KL}{r}\right)_z = 103 \quad \text{or} \quad \frac{F_y}{F_{ez}} = \frac{50}{27.21} = 1.837$$

$$\left(\frac{KL}{r}\right)_{F-T} = 52 \quad \text{or} \quad \frac{F_y}{F_{eF-T}} = \frac{50}{74.90} = 0.667$$

$$F_{crF-T} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y = 37.81 \text{ ksi} \qquad \text{(AISC E3-2)}$$

$P_n = A_g F_{cr} = 165.20$ kips $\therefore$ Therefore, the axial buckling capacity about the Z-Z axis controls.

3.3 Double-Angles

The ANSI/AISC 360-10, Specification for the Design of Steel Buildings (AISC 2010) Chapter E, Design of Members for Compression and more specifically, E6. Built-up Members govern the design of two or more shapes (i.e., in this case two equal or unequal leg angles interconnected with bolts or welds). The Specification states,

"The nominal compressive strength of built-up members composed of two shapes that are interconnected by bolts or welds shall be determined in accordance with Sections E3, E4, or E7 subject to the following modification. In lieu of more accurate analysis, if the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, KL/r is replaced by $(KL/r)_m$ determined as follows:

(a) For intermediate connectors that are bolted snug-tight,

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad \text{(AISC E6-1)}$$

(b) For intermediate connectors that are welded or are connected by means of pretensioned bolts,

$$\text{(i) When } \frac{a}{r_i} \le 40 \quad \left(\frac{KL}{r}\right)_m = \left(\frac{KL}{r}\right)_o \qquad \text{(AISC E6-2a)}$$

$$\text{(ii) When } \frac{a}{r_i} > 40 \quad \left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{K_i a}{r_i}\right)^2} \qquad \text{(AISC E6-2b)}$$

where

$\left(\frac{KL}{r}\right)_m$ = modified slenderness ratio of built-up member

$\left(\frac{KL}{r}\right)_o$ = slenderness ratio of built-up member acting as a unit in the buckling direction being considered

K_i = 0.50 for angles back-to-back; 0.75 for channels back-to-back; 0.86 all others

a = distance between connectors, in.

r_i = minimum radius of gyration of individual component, in.

What is known about these types of compression members (see Fig. 3.2) is:

- Flexural buckling will occur in the plane of symmetry
- Singly-symmetric sections can buckle either in a flexural or flexural-torsional mode
- For singly symmetric sections such as double angles, which are commonly used as truss members and bracing members, flexural buckling about the x axis or flexural-torsional buckling about the y axis can occur.

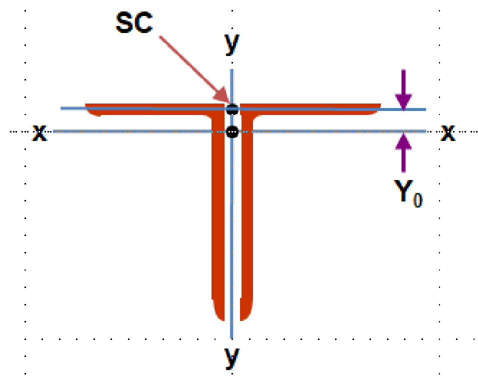


Figure 3.2 Double Angles – Equal or Unequal Leg Properties

3.4 Double-Angle Bracing Member Design Example

Determine the design strength of a 14 ft. diagonal bracing member comprised of 2L6 x 4 x 5/8 LLBB attached to 3/8" thick gusset plates, i.e., gap between long legs = 0.375 in. There are two intermediate fully tightened (i.e., pretensioned) bolts at 56 in. spacing.

Use K = 1.0.

Material Properties: F_y = 36 ksi E = 29,000 ksi G = 0.385E

Single Angle Section Properties:

r_z = 0.859 in. J = 0.755 $in.^4$
r_{yi} = 1.13 in. C_w = 1.59 $in.^6$

Double Angle Section Properties:

A = 11.7 $in.^2$ r_x = 1.89 in.
I_x = 42.0 $in.^4$ r_y = 1.66 in.
H = 0.684 in. $\bar{r}_o$ = 3.05 in.

$\left(\frac{KL}{r}\right)_x = 88.4$ $\left(\frac{KL}{r}\right)_y = 101.2$ $\frac{a}{r_z} = \frac{a}{r_i} = \frac{56}{0.860} = 65.1 > 40$

Note: Design strength neglecting Flexural-Torsional buckling; Y-Y axis is built-up and acts as a single unit, therefore,

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} = \sqrt{(101.2)^2 + (65.1)^2} = 120.34$$

F_e = 19.76 ksi

F_y/F_e = 36/19.76 = 1.82 < 2.25

$F_{cr} = 0.658^{\frac{F_y}{F_e}} F_y = 16.79$ ksi

$P_n = A_g F_{cr}$ = (11.7)(16.79) = 196.44 kips

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{K_i a}{r_i}\right)^2} = \sqrt{(101.2)^2 + \left[(0.5)\left((65.1)\right)\right]^2} = 106.30$$

F_e = 25.33 ksi

F_y/F_e = 36/25.33 = 1.42 < 2.25

$F_{cr} = 0.658^{\frac{F_y}{F_e}} F_y$ = 19.86 ksi

$P_n = A_g F_{cr}$ = (11.7)(19.86) = 232.36 kips ∴ USE

$$F_{cr} = \frac{F_{cry} + F_{crz}}{2H}\left[1 - \sqrt{1 - \frac{4F_{cry}F_{crz}H}{\left(F_{cry} + F_{crz}\right)^2}}\right] = \frac{19.76 + 151.47}{2(0.684)}\left[1 - \sqrt{1 - \frac{4(19.76)(151.47)(0.684)}{(19.76 + 151.47)^2}}\right]$$

F_{cr} = 18.91 ksi

$P_n = A_g F_{cr}$ = (11.7)(18.91) = 221.22 kips

ASD $\frac{P_n}{\Omega} = 132.47$ kips LRFD $\phi P_n = 199.10$ kips

Summary: The Flexural-Torsional buckling load is ~5% less than the Flexural buckling load in this case. If the angle legs are thin (slender) the buckling capacity could be up to 20% less than the Flexural buckling load.

4. Analysis Inputs and Models

4.1 Model Descriptions

The finite element analysis (FEA) models were developed using the commercially available general purpose structural analysis software SAP2000 (CSI 2002), as well as the non-commercial software MASTAN2. Both models utilize line or beam elements with cross section properties consistent with Figure 4.1a. The columns are continuous from the base to the top of the structure, while all beam and bracing elements have moment releases at each end. This approach matches the idealized conditions that are often assumed in design practice for braced frame structures and is considered a reasonable approach for the braced frame configuration under study, which involves double angle bracing elements, and wide flange columns and beams. All beam elements were modeled with 8 divisions along their length.

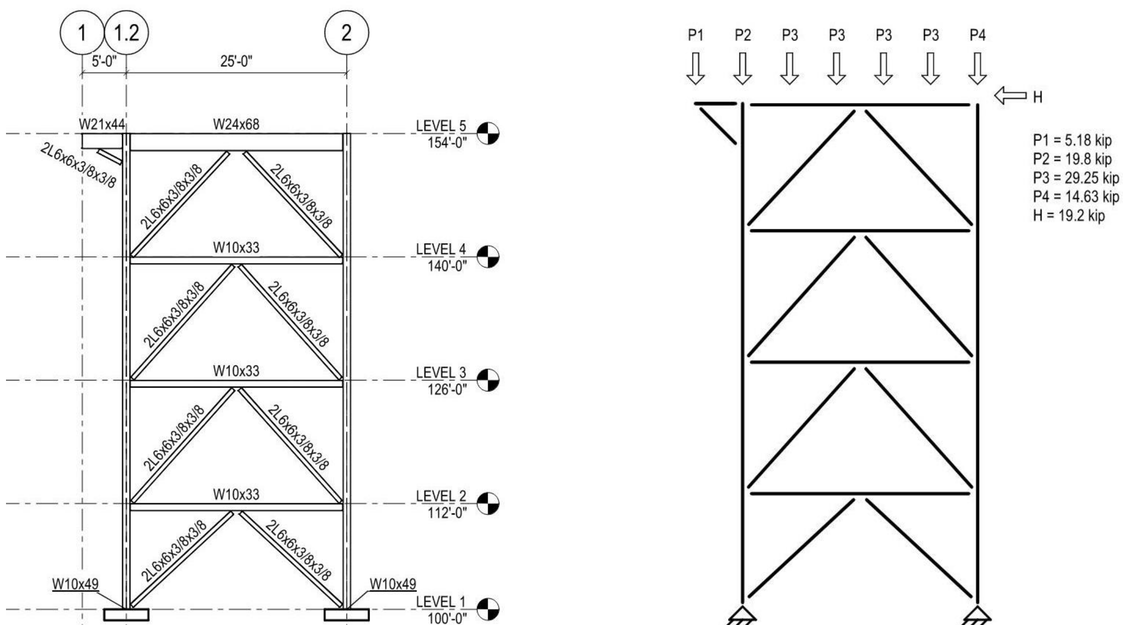
A major disadvantage of using line elements is that local effects at connections are not captured. Thus, gusset plates and connection details are not accounted for in this model. Also, cross section distortion of individual connection components or framing members is not captured. These modeling choices tend to overestimate the stiffness of the braced frame as local deformations are not captured. However, providing moment releases at all beam and brace elements tends to underestimate the stiffness of the braced frame.

Boundary conditions are idealized, similar to what is done in typical design practice. The models assume plane frame behavior, that is, no out-of-plane behavior is considered. This is reasonable given that a braced frame occurs along both orthogonal column lines, and the upper-most platform level provides lateral restraint to the framing at that level. Additionally, the base connections of the columns to the foundation piers are idealized as pin-ended connections as the flexural stiffness of the base plate connections is relatively small compared to the stiffness of the column.

Material properties used in design are ASTM A992 steel for all wide flange elements with F_y equal 50 ksi, and ASTM A36 steel for all angle and plate elements with F_y = 36 ksi.

Only a single loading case is considered. Refer to Section 3.1 for a general overview of the loading conditions. For these models, a tributary width of 22'-6" for gravity loading onto the frame was used. Dead loads considered a loading of 50 psf, and a live load of 125 psf was used, except at the portion of the platform between Grids 1 and 1.2 (Refer to Figure 4.1a) which considered a 20 psf live load. A single lateral live load of 12 kip was included. Finally, member self-weight was included. These loads are factored consistent with LRFD design.

Other global effects in the temporary work platforms are not captured in the FEA model. For instance, instabilities of other framing elevations, or any torsional effects of the overall structure are not captured. Member instabilities at other locations, for instance, lower level beams which are not directly taking gravity load, but act to laterally brace vertical bracing elements is not considered.



(a) (b)

Figure 4.1: Critical Frame Geometry and Loading
(a) Frame Geometry and Member Sizes used in Analysis
(b) Loading and Boundary Conditions used in Analysis

4.2 Analysis using the Direct Analysis Method

Multiple methods are available to design for stability of a steel frame. Any stability design is a combination of analysis to determine the required strength and stiffness at the component level, while ensuring the overall and local stability effects are accounted for. In the case of a braced frame like the one being studied in this paper, all stability design methods presented in AISC (2010, 2016) are available including the Effective Length Method. It is common in design practice to use the Effective Length Method for a braced frame as it reduces the rigor and time to perform the analysis. Note that AISC Commentary in Appendix 7 states "In braced-frame systems, shear-wall systems, and other structural systems where lateral stability and resistance to lateral loads does not rely on the flexural stiffness of columns, the effective length factor, K, of members subject to compression shall be taken as unity unless a smaller value is justified by rational analysis.". However, in our case, we will use the Direct Analysis Method (DM) to better understand 2^{nd} order effects on the structure, although they are likely to have a small impact because braced frames typically have relatively high lateral stiffness compared to the overall gravity loads being supported.

Any stability design must account for all deformations, consider second order effects, geometric imperfections, stiffness reduction due to inelasticity, and uncertainty in strength and stiffness. The Direct Analysis Method captures these items in the overall structure response through the use of Notional Loads, a stiffness reduction factor of 0.8E, and an additional stiffness reduction τ_a for members with high levels of compressive loads. From there, impacts at the member strength level are captured in the AISC design equations; refer to Section 3 for an overview of single and double angle provisions.

SAP2000 software is capable of performing a DM analysis, and a code check based on the AISC provisions. Results from the DM analysis including ultimate axial loads, and demand/capacity ratios are shown in Figure 4.2 It is worth noting that the ratio of 1^{st} order to 2^{nd} order lateral drifts is 1.01 indicating relatively minor amplifications in the frame drift due to second order effects. Note in Figure 4.2 that the controlling member design is the upper-most double angle brace component. SAP2000 results are consistent with AISC Chapter E and F. Axial loads dominate the design of the double angle, although there is a small amount of flexure associated with self-weight of the member, as well as P-δ effects. The design for compression considers a fully composite section associated with welded intermediate connectors.

To address the matter of this paper, the impacts of not having connectors between each angle the same axial loads and moment are used but member strength equations based on single angle capacities are used instead. SAP2000 provides design code checks about the single angle geometric axes, so member strengths are calculated separately using the provisions of Section E5. The axial demands and moments are split equally between the two angle members, and the results provide a demand/capacity ratio of 1.38, exceeding code permitted levels.

4.3 Analysis using AISC Appendix 1

Appendix 1 is an extension of the Direct Analysis Method in which member design provisions are provided for explicitly by modeling member imperfections and inelasticity. MASTAN2 is used for this analysis. MASTAN2 allows for modeling of member yield strength to capture failure modes, and uses a modified tangent modulus stiffness approach to capture nonlinearity associated with partial yielding due to residual stresses. Member out-of-straightness is modeled directly accounting for an assumed L/1000 out-of-straightness. Out-of-plumbness is also modeled directly instead with an assumed H/500 out-of-plumbness. This approach is capable of capturing flexural buckling as well as cross-sectional yielding due to combined axial and flexural stresses, but cross-section slenderness and flexural-torsional buckling are not captured. However, these limits states do not control or influence the behavior of the frame, and so excluding them from the analysis is reasonable.

Results of the MASTAN2 analysis results for the double angle brace are shown in Figure 4.4. It should be noted that at a Load Ratio of 1.0 the axial and flexural demands for the critical bracing member are comparable to the results from the Direct Analysis Method presented in Section 4.2.

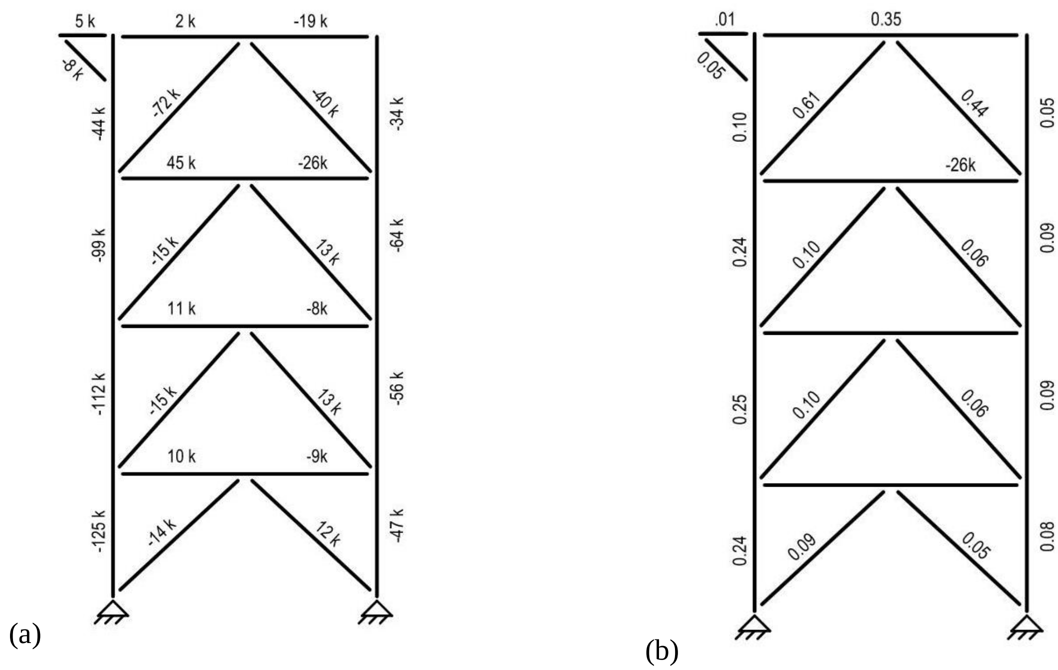


Figure 4.2: Design Results for Double-Angle Model
(a) Frame Ultimate Axial Loads from Analysis
(b) Frame Demand/Capacity Ratios from Analysis

Similarly, the MASTAN2 models are created to capture the behavior of the non-composite double angle model. However, modifications to the member properties are required to account for the differing behavior of a single angle compression element when compared to a double angle compression element. The question to answer is this: what limit states do the AISC provisions account for, and how can we include these limits states directly in our analysis? Section 3.1 provides an overview of single angle compression capacity and presents introduces modified effective slenderness ratio approach that is used. This modified slenderness ratio captures both flexural buckling about the principal minor axis, and also the effects of combined bending and compressive stress about a geometric axis of the unconnected angle leg. In our case the slenderness ratio is relatively large ($KL/r_x = 120$), so that will be the focus of this discussion. Since the AISC design equations account for eccentricity at the connection, the impacts of eccentricities in our analysis need not be captured directly for the sake of member strength design. There is still the matter of overall frame stiffness. It is true that member P-δ effects from these eccentricities will impact member stiffnesses; however, because we have braced frame geometry this impact is minor as the axial stiffness of the brace is still relatively large. Essentially, lateral stiffness of the frame will be fairly constant until the limit state of flexural buckling of the single angle member is achieved in which case the frame will be considered to have reached its limit load.

To capture the modified slenderness ratio within the analysis model, the single angle section properties are modified. The member moment of inertia is modified by the ratio $\frac{r_{AISC}^2}{r_x^2}$, where r_x is the member radius of gyration about the geometric axis, and r_{AISC} is the modified radius of gyration per equations E5-1 and E5-2 from the AISC Specification (AISC 2016).

As a proof of concept, this approach is used for a single angle L6x6x3/8 A36 steel element with varying L/r_x values. Results from MASTAN2 Appendix 1 analyses are presented alongside design capacities from AISC. Note that two models are created in MASTAN2 to compare to AISC, the first model considers the plastic section modulus of the angle, Z_x, to be equal to the plastic section modulus about the geometric axis of the angle. The second modifies the plastic section modulus of the angle to be equal to the elastic section modulus, S_x. Results show that the approach described above provides angle axial capacities that are within 0% to 6% of the design axial capacities from AISC. Both approaches provide reasonable accuracy compared to the AISC provision capacities. However, using the elastic modulus yields slightly more conservative results. Therefore, the elastic section modulus approach is used in the planar frame analyses.

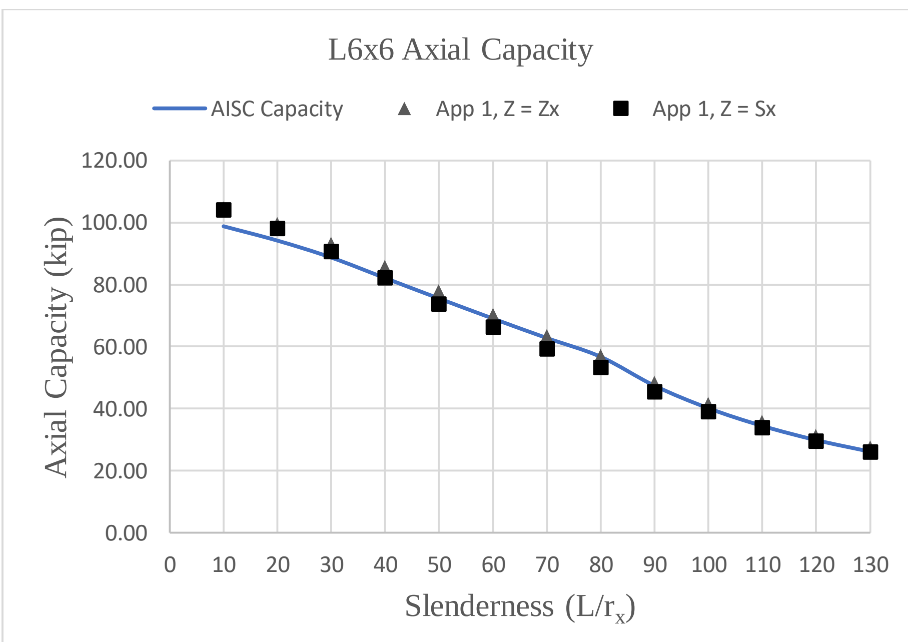


Figure 4.3: Comparison of AISC Single Angle Axial Capacity to MASTAN2 Appendix 1 Values with Various Plastic Section Modulus Values.

With the modeling approach confirmed, the MASTAN2 planar frame single and double angle model analyses are presented in Figure 4.4. Figure 4.4 shows the frame lateral deflection plotted with the load ratio, consistent with the loads described in Section 4.1. The maximum load ratio for the double angle frame is 1.51, while the maximum load ratio for the single angle frame is 0.63. The peak load is associated with the maximum axial capacity of the critical bracing member.

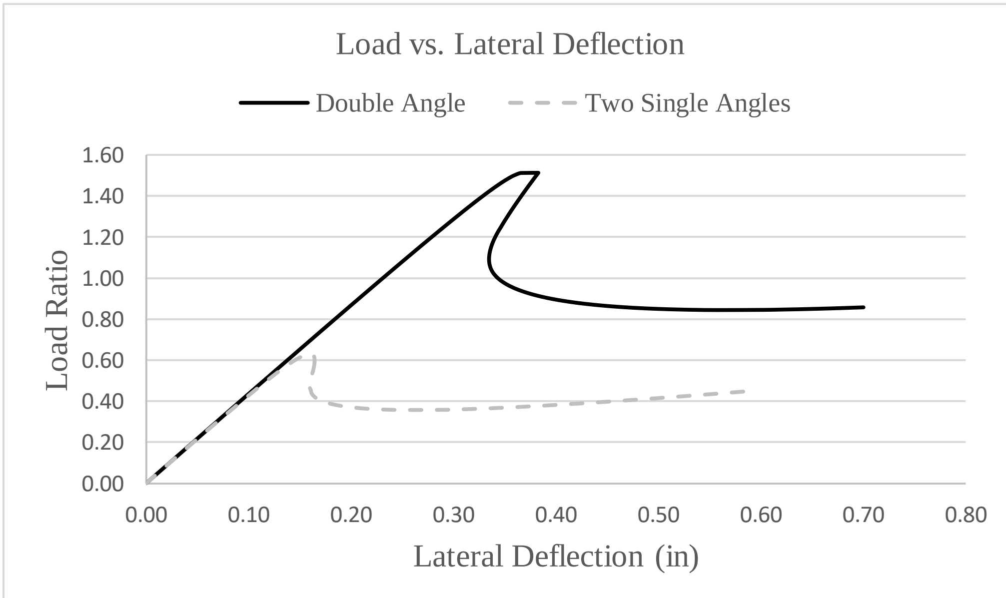


Figure 4.4: Compression Member Behavior: Built-up Double Angle vs. Two Single Angles

5. Discussion

5.1 Analysis Results

Results from the SAP2000 DM analysis and design show that the double-angle bracing configuration has significantly more capacity than the two single angle brace configuration. This is intuitive based on the member strength provisions from the AISC Specification. Similarly, the results of the MASTAN2 Appendix 1 analysis show similar results. A comparison of the two methods is shown in Table 5.1 where the equivalent load ratio or demand to capacity ratio is provided. The Appendix 1 results provide lower capacities associated with the frame compared to the Direct Analysis Results.

Table 5.1 Comparison of Results of Direct Analysis Method and Appendix 1

Method	Software	Model	Demand to Capacity Ratio	Load Ratio
Direct Analysis Method	SAP2000	Double Angle	0.61	1.64
Appendix 1	MASTAN2	Double Angle	0.66	1.51
Direct Analysis Method	SAP2000	Two Single Angle	1.38	0.72
Appendix 1	MASTAN2	Two Single Angle	1.59	0.63

Therefore, based on the above results, without intermediate bolted or welded connectors between the two angles, the frame(s) did not have adequate capacity to carry the specified loading.

5.2 Understanding Today's Processes

5.2.1 Understanding today's process of design and analysis

When an Engineer of Record (EOR) designs a structure (temporary or permanent) the structural analysis, i.e, loads and load combinations is typically performed using commercially available software. The first step in using any software is the creation of a structural analytical model. The EOR, or someone working on behalf of the EOR, uses their judgment to capture structural behavior from the proposed physical structure. This involves studying the proposed structure as far as geometry, layout, and member shape types, and based on their experience (judgment) creating an analytical model that represents the physical structure. Additionally, the type of analysis to be performed, e.g., elastic or inelastic, and the loads and load combinations to be used need to be input by the Designer. For a structure like the temporary work platforms being discussed, the members will be represented by a 6 DOF beam element, and an elastic analysis will be performed.

The results of the elastic analysis will be used to determine nodal displacements, base reactions, and member forces. In addition to providing the analysis results – member forces and moments, today's software will typically make all the required "Code Checks" for a given Specification, e.g. AISC 360-10. More sophisticated software programs will even provide preliminary connection designs. The Designer will often review the steel code check preferences or defaults prior to performing the code checks. Various items are weighed in this process, for instance: the experience of the Designer, office preferences, industry standards of practice, and experience with a specific software package. Often, program defaults may be used unknowingly. Through this process, items such as the design methodology (ASD vs. LRFD), member effective unbraced lengths, etc. will be determined that are crucial inputs to the design of the structure. If all members in the structure pass their required code checks, then the design can be detailed to include all the member sizes, including their connections. The culmination of this process results in Engineering Drawings that are provided to the steel fabricator for use in detailing.

5.2.2 Understanding today's process of detailing for fabrication

The Engineering Drawings are used in the creation of a Structural Steel Detailing model. The intent of the Engineer must be fully represented in their drawings, as any results of the analysis software are typically not provided to the Fabricator or Detailer. With this being the case, how does the Detailer know the specifics of a built-up member that was selected and confirmed in a computer model? This is where knowledge of the Specification, especially our project's code of record, AISC 360-10 Chapter E, DESIGN FOR COMPRESSION, Section E6, BUILT-UP MEMBERS is very important. Subsection E6.1 Compressive Strength states,

> "This section applies to *built-up members* composed of two shapes either (a) interconnected by bolts or welds, or (b) with at least one open side interconnected by perforated *cover plates* or *lacing* with *tie plates*. The end *connection* shall be welded or connected by means of *pretensioned bolts* with Class A or B *faying surfaces*."

The Subsection goes on to state,

> "The *nominal compressive strength* of built-up members composed of two shapes that are interconnected by bolts or welds shall be determined in accordance with Sections E3, E4 or E7 …"

5.2.3 Understanding today's procurement, subcontract contractual process

For this case, AISC 360-10 Chapter E, Section 6.1(a) above should have applied, but was missed going from the computer model to the Engineering Drawings to the Detailing model of the temporary work platform structure. And, based on what was eventually discovered, what was missed being placed on the Detailing or "Shop" drawings was then missed during fabrication. Since these intermediate connectors were not shown on the fabrication drawings there was no anticipation that there needed to be any interconnection provided to create each double angle built-up member called for on the erection drawings. Since these were not shown on the drawings, the Foreman in charge of erecting the Temporary Structural Steel Work Platform(s) and the Field Engineer assigned to oversee the erection of the Platform(s) would not have questioned how these members were supposed to be "put together" other than by using the drawings provided in the Construction Work Package.

5.2.4 Understanding today's work process and relying on commercially available software approved for project use

The last point that is needed to be made has to do with the commercial software used for the structural analysis and subsequently the software package used for detailing. One does not have to be an expert in the use of every piece of commercial software available for use, but one does have to know something about assuring that the member selections allowed are properly detailed for fabrication. Just because the analysis software package allows one to select a member that is considered a built-up member such as a double-angle member it does not assume to know how the Engineer "driving the software" envisions how the double-angle member is to be fabricated. In today's steel detailing software packages it would seem that one needs to be knowledgeable enough to stop and question the EOR when something like intermediate connectors are not called out. This can be an error prone situation, and as just happened with this design and fabrication of several Temporary Work Platforms, it was declared a dangerous, unsafe condition and was cause for immediate action to be taken to close the platforms that had been erected until the design, detailing, and fabrication was brought into alignment with the AISC 360-10 Specification.

6. Conclusions

Industrial structures such as temporary work platforms often use double angle bracing members. Double angle member design must carefully consider the use of intermediate connectors, as without them the member must be analyzed and designed as two separate single angles. The analysis showed that both the Direct Analysis Method and Appendix 1 methodology is well suited for this type of problem. Conservative approximations for the single angle stiffness may be made to capture code endorsed flexural buckling capacities. This paper showed that for the work platform being studied that the absence of intermediate connectors impacts the structural stability of the member, and thus the overall structure, and as a result the overall load carrying capacity of the work platform.

References

AISC (2010). “ANSI/AISC 360-10 Specification for Structural Steel Buildings.” American Institute of Steel Construction, Chicago, IL.

AISC (2011). **Steel Construction Manual**. Fourteenth Edition, American Institute of Steel Construction, Chicago, IL.

AISC (2016). “ANSI/AISC 360-16 Specification for Structural Steel Buildings.” American Institute of Steel Construction, Chicago, IL.

ASCE, SEI/ASCE 7-10 (2010). “Minimum Design Loads for Building and Other Structures.” American Society of Civil Engineers, Reston, VA.

ASCE, SEI/ASCE 37-14 (2014). “Design Loads on Structures During Construction.” American Society of Civil Engineers, Reston, VA.

ASTM A36/A36M-14 (2014). “Standard Specification for Carbon Structural Steel.” American Society of Testing and Materials, ASTM International, West Conshohocken, PA.

ASTM A307-14 (2014). “Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod 60 000 PSI Tensile Strength.” American Society of Testing and Materials, ASTM International, West Conshohocken, PA.

ASTM A992/A992M-11 (2011). “Standard Specification for Structural Steel Shapes.” American Society of Testing and Materials, ASTM International, West Conshohocken, PA.

ASTM F1554-15 (2015). “Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength.” American Society of Testing and Materials, ASTM International, West Conshohocken, PA.

ASTM F3125/F3125M-18 (2018). “Standard Specification for High Strength Structural Bolts and Assemblies, Steel and Alloy Steel, Heat Treated, Inch Dimensions 120 ksi and 150 ksi Minimum Tensile Strength, and Metric Dimensions 830 MPa and 1040 MPa Minimum Tensile Strength.” American Society of Testing and Materials, ASTM International, West Conshohocken, PA.

AWS D1.1/D1.1M:2015 (2015). “Structural Welding Code-Steel.” American Welding Society, Miami, FL.

IBC (2012). **International Building Code.** International Code Council, Country Club Hills, IL.

RCSC (2009). “Specification for Structural Joints Using High-Strength Bolts.” Research Council on Structural Connections, c/o AISC, Chicago, IL.

CSI (2002) SAP 2000 Analysis Reference Manual. Computers and Structures Inc., Berkeley, CA.

Lutz, L. A. (2017). “The History of the Development of the Single Angle Design Provisions in the AISC Specification.” Proceedings of the Annual Stability Conference, Structural Stability Research Council, San Antonio, Texas, March 21-24, 2017.

MASTAN2, Version 3.5, retrieved from http://www.mastan2.com/about.html

Sato, A. and Uang, C-M. (2007). “Modified Slenderness Ratio for Built-up Members.” Engineering Journal, AISC, Vol 44, No. 3, pp. 269-280.

Sherman, D. R. and Yura, J. A. (1998), “Bolted Double Angle Compression Members,” *Journal of Constructional Steel Research*, Vol. 46, No. 1-3.

Ziemian, R. D., Editor (2010). **Guide to Stability Design Criteria for Metal Structures**. Sixth Edition, John Wiley & Sons Inc., Hoboken, NJ.