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Stability Considerations for Concrete Forming Support Systems

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Abstract
The contractual responsibilities of building designers are often limited to the performance of a structure after construction is complete. That is, the designers leave the means and methods of construction, including the stability of the partially completed structure, to the contractor. This can lead to problems, as contractors are likely not trained in structural stability and may not understand how seemingly minor alterations to components can have a disproportionate effect on the stability of the temporary works. This paper presents a case study that explores the stability of shoring systems used to support concrete formwork, and in particular the adverse effects of modifying component lengths and the addition of aluminum spacers. Classical calculations are performed and verified using OpenSees analyses to determine the expected change in capacity due to the adjustments. Finally, we offer advice to engineers and contractors warning of dangerous adjustments so they might avoid similar damage or collapse of forming support systems on their projects.

1. Background

Aluminum shoring systems for cast-in-place concrete typically consist of braced, prefabricated aluminum frames with screw jack leg extensions supporting aluminum stringers and joists on which temporary wood forming is erected. They provide versatile, off-the-shelf, often proprietary solutions for contractors for several reasons:

- The shoring system can be easily adjusted in-situ to create steps in the concrete profile.
- The systems have a high strength-to-weight ratio and can be reused multiple times.
- Labor is minimized by not requiring measuring and cutting of lumber to match varying profiles.

Despite their many advantages, the relative slenderness of the aluminum supports make them susceptible to buckling failures if not properly braced or if the systems are modified beyond their design limits.

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1.1 The Failure and Investigation

In 2014, a parking deck under construction experienced a partial collapse during a concrete pour. The system had been shored using aluminum shoring that supported wood formwork. Figure 1 and Figure 2 show the collapse and a close-up of the shoring and formwork, respectively.

Figure 1: Partial garage collapse during concrete pour

Figure 2: Close-up of aluminum shoring/forming system

The authors investigated the partial collapse to identify the fundamental cause(s). The investigation was complicated due to now-hardened concrete concealing a significant portion of the site and evidence. The site was meticulously cleared, photographs were taken, and the relevant evidence was collected and preserved. It was ultimately determined based on the evidence collected and analyses that instability of the aluminum shoring systems caused the collapse. Two types of analysis were used to assess the shoring/forming system utilizing the collected data: 1) an initial assessment of the shoring towers using Euler buckling models with reasonable boundary assumptions, and 2) a refined load-deflection analysis that modeled individual framing boundary conditions explicitly. Sections 2 and 3 below detail both the Euler and load-deflection analyses.

1.2 Construction Documents

The construction documents specified six-foot tall prefabricated shoring tower frames, stacked two high, with adjustable leg extensions. According to calculations performed by the shoring designer, the unfactored design load on each shoring tower leg was 11.7 kips and was checked against an allowable load of 13.8 kips per leg based on a maximum 30-inch leg extension (a total tower height of 17 feet). Allowable loads (published in manufacturer's product specifications) were based on a factor of safety of 2.5, so the ultimate capacity of the leg was estimated at 34.5 kips.

Site evidenced indicated that several of the prefabricated frames in the area where the collapse initiated were only five feet tall. As a result, these shorter frames required longer leg extensions to achieve the same soffit height. An equal-height tower using two five-foot tall frames would require 42-inch leg extensions, but the maximum leg extension was 39 inches; additional shimming would have been required. The published allowable load for 39-inch leg extensions on five-foot tall frames was 9.8 kips (24.5 kips ultimate) which is 29% lower than the allowable load assumed in the design using six-foot tall towers, and 16% lower than the unfactored design load. Table 1 summarizes the loads as reported by the manufacturer for the five-foot and six-foot tall tower frames.

Table 1: Manufacturer's capacity of various frame-heights

Unfactored Design Load = 11.7 kips		
Frame Size	Allowable Load (kips)	Ultimate Load (kips)
6' (as-designed – 30" extensions)	13.8	34.5
5' (as-built – 39" leg extensions)	9.8	24.5

Why did the failure occur if the ultimate capacity of the shorter frames was greater than the unfactored design load calculated by the shoring designer?

2. Simplified Analysis

A simplified analysis was first performed to determine the bounds of the frames' buckling capacities using Mastan2 (2015) and hand calculations. Assumptions and techniques required to perform the analysis concerning the frame components, boundary conditions, and loading are discussed herein.

2.1 Frame Components

The frames were constructed of two hollow aluminum "column" sections spaced approximately six feet apart and connected with diagonal and horizontal bracing. As discussed previously, the height of the frame segments was either five or six feet. The top and bottom of the frame column sections were either spliced into another frame or incorporated a threaded leg extension. The leg extensions were adjusted to fit snugly between the existing slab below and the concrete formwork above. Figure 3 is a schematic out-of-plane elevation of a tower frame assembly shown with cross-bracing between the two sides. There were a number of additional components that add to the total height of the frame assembly and are shown in Figure 4.

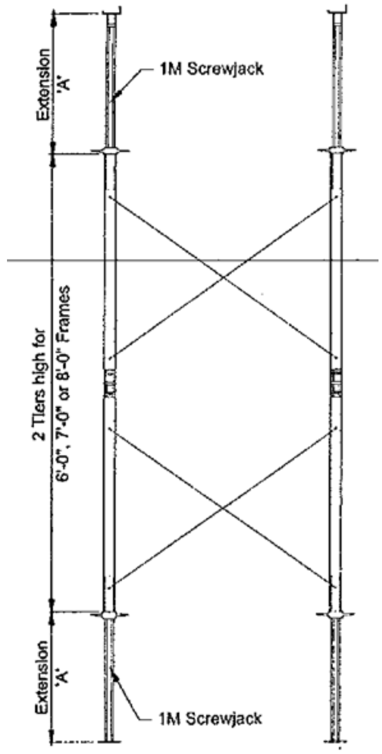

Figure 3: Tower model

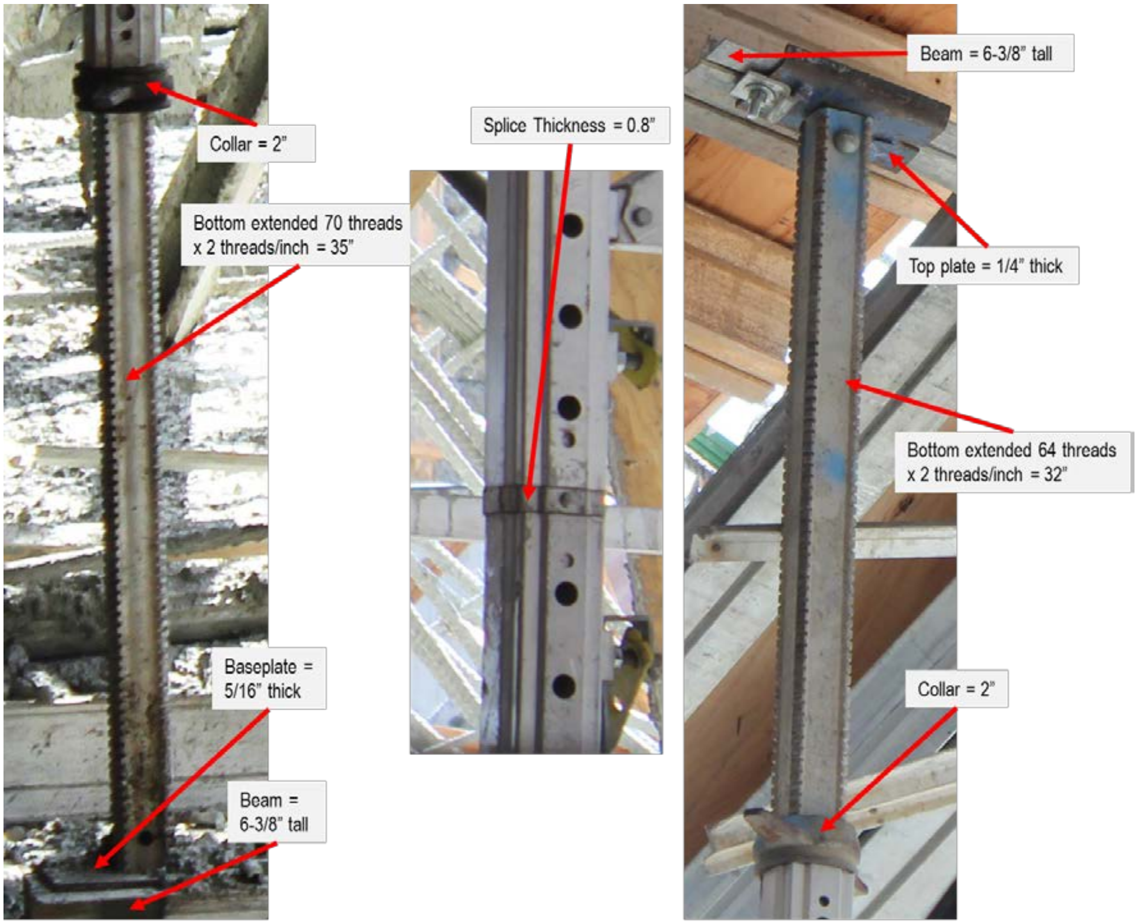

Figure 4: Frame components

The frame and leg cross section were subdivided into many fibers (based on field measurements) to determine the aggregated cross-sectional properties for analysis. Section properties were calculated directly for the horizontal, diagonal, and out-of-plane braces. The cross-sections and their tabulated properties are presented in Figure 5 and Table 2, respectively.

At the underside of the formwork, there were stringers that supported the joists and forming. The stringers were supported by the upper leg extensions as shown in Figure 4. In some instances, there were rows of stringers placed on the hardened slab below the frames to provide a base for the tower leg extensions. Such stringers were not shown on design drawings and were likely used by the contractor as a workaround during the construction phase, potentially to compensate for using a five-foot tower height in particular areas of the project. The stringers are shown in cross-section in Figure 6, including one such example showing fracture and local buckling of the stringer web.

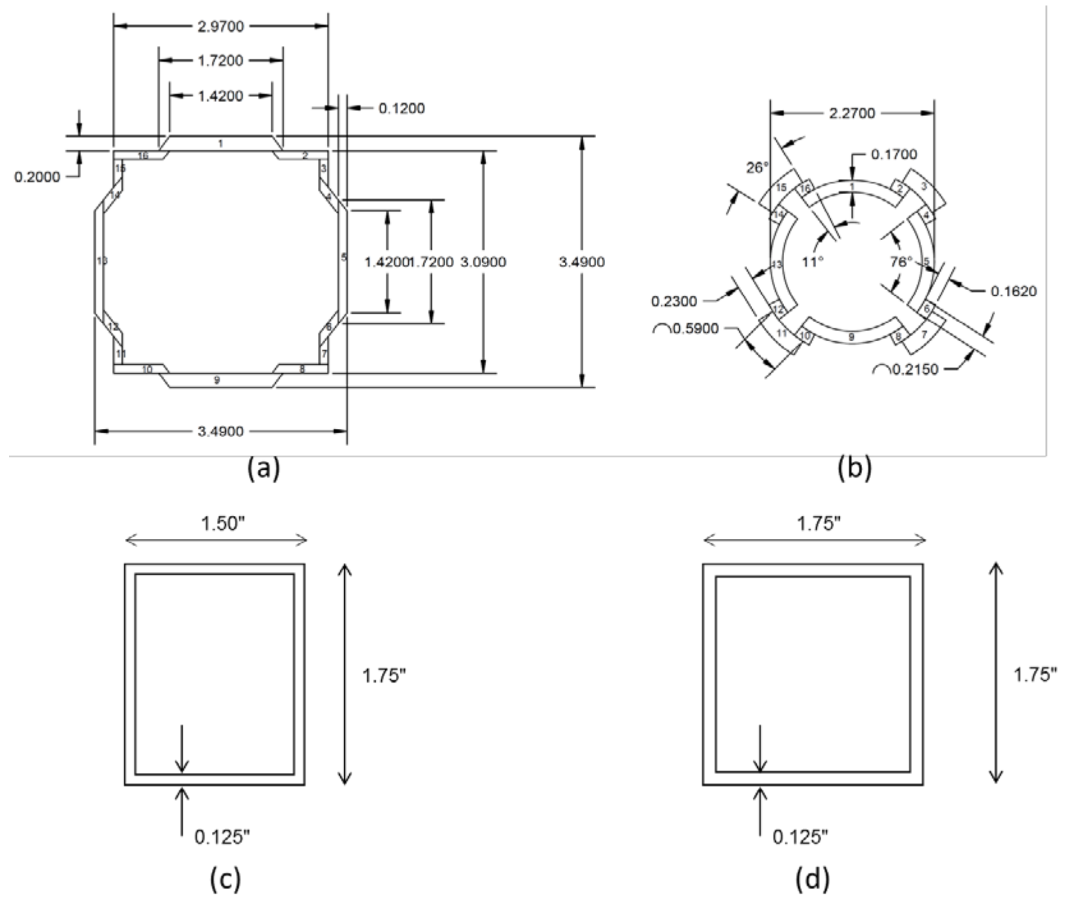

Figure 5: Frame cross-sections: a) column, b) leg extension, c) horizontal brace, and d) diagonal brace

Table 2: Frame and leg cross-sectional properties

Segment	Section Properties		
	A (in.^{2})	I_{xx} (in.^{4})	I_{yy} (in.^{4})
Frame Column	1.71	2.97	2.41
Leg Extension	1.84	1.39	1.39
Horizontal Brace	0.75	0.32	0.25
Diagonal Brace	0.81	0.36	0.36
Out-of-plane Brace*	0.18	0.02	0.02

*assumed pipe 1” dia. x 1/16” thick

Figure 6: Aluminum stringers; left example shows web fracture and buckling

2.2 Boundary Conditions and Member Releases

The actual boundary conditions were idealized for an initial Euler buckling assessment of the tower as either fix-fix or pin-pin. These models ignore the presence or absence of stringers, which are accounted for in the refined analyses discussed in Section 3.

The separate tower column components (top extension, top frame, bottom frame, bottom extension) are modeled as continuous. The planar and out-of-plane bracing is assumed to only transmit axial forces (moment and torque are released) at the bracing components' juncture with the frame columns.

2.3 Loading

The load path onto the tower components was as follows: wet concrete → formwork → joists → stringers → upper tower leg extension → top tower frame → bottom tower frame → lower tower leg extension → stringers (if present) → hardened slab. In all of these steps, it was assumed that the load is applied concentrically about the principal axis of the member in question. That is, the tower components were assumed to have the load applied through the centroid of their cross-section. The stringers were assumed to be loaded through their vertical axis when viewed in cross-section.

For the Euler buckling analyses, a unit point load was applied at the top of the top frame extension at each of the four tower columns. An elastic Eigenvalue analysis is performed in Mastan2 and the program-reported load multiplier provides the load on each tower frame column at incipient buckling.

It should be noted that the assumption about concentric loading of the stringers does not hold true for all observations as evidenced by Figure 7 showing a base plate that is offset from the center of either of the two stringers below. While the loading offset effects the buckling capacity of the stringers, the effect is considered negligible for these analyses.

Figure 7: Offset in stringer loading

2.4 Results

An elastic buckling analysis of the tower assembly (excluding stringers) was performed and the results are shown in Table 3. The results indicate that in-plane buckling controls for both the pin-pin and fix-fix boundary conditions models. The multiplier between the pin-pin and fix-fix boundary condition models is around 3.7 for both in-plane and out-of-plane buckling.

Table 3: Buckling capacity of shoring towe (without stringers)

Buckling Direction	**Boundary Condition**	**P_{cr} per column (kips)**
In-plane	Pin-Pin	15.8
	Fix-Fix	58.4
Out-of-plane	Pin-Pin	18.4
	Fix-Fix	68.4

The controlling condition when considering all components of the tower frame system was the in-plane buckling capacity of 15.8 kips which is 35% more than the unfactored design load calculated by the shoring designer.

In order to assess any impact the stringers may have on the buckling capacity of the shoring tower system, initial hand calculations were performed to determine an approximate capacity of the stringers. Three limit states were considered to try and bound the likely capacity of the stringer web when subject to compression:

1. Elastic buckling – this failure mode assumes a model of the web only that is clamped along its top and bottom edge. As the concentrated load is applied to the flange and transmitted to the web, the load will fan out through the depth of the web. The effective stringer length used to calculate the plate's moment of inertia is assumed as 4, 8, 12, and 16 inches. The corresponding values of P_{cr} are listed in Table 4 for the stringer web with these various effective lengths.
2. Web local yielding – this failure mode is intended to limit yielding through the web with an assumed stringer length of 8 inches.
3. Web crippling – this failure mode is intended to predict buckling of the web into several waves directly beneath the load with an assumed stringer length of 8 inches. For more slender webs, this is predicted to occur prior to web local yielding (AISC 360, Pp. 16.1-417).

The results of the hand calculations presented in Table 4 indicate that for a stringer length equal to 8 inches, the simplified elastic buckling model of the web would control the design of the stringer at a capacity of 44.4 kips versus 100 kips for web local yielding or 47 kips for web local crippling. However, as mentioned previously, the idealized boundary conditions for this analysis were perfect fixity provided by the flanges, and thus the capacities of the stringer webs in Table 4 could be considered upper bounds to the true capacity.

Table 4: Buckling capacity of stringer webs

Failure Mode	**Effective *L* (in.)**	**P_{cr} per column (kips)**
Elastic buckling	4	22.2
	8	44.4
	12	66.6
	16	88.8
Web Local Yielding	8	100
Web Local Crippling	8	47.0

3. Refined Analysis

In the next step, a refined computer model was created to corroborate the anticipated failure mode and expected maximum tower capacity. An OpenSees model was created and analyzed given the assumptions and modeling techniques detailed in the subsequent sections.

3.1 Software Description

OpenSees (version 2.5.0) was used to model the tower frame system (2013). Primarily intended for earthquake engineering simulation, the Open System for Earthquake Engineering Simulation is an object oriented, open source structural software framework. OpenSees is research-grade software with significant contributions from researchers around the world. The structural framework allows users to expand the extensive element and material libraries. The program performs a load-deflection analysis where instabilities are modeled via large-displacement local-to-global force transformations based on a co-rotational formulation developed by Souza (2000).

Each element was discretized into fifteen elements with five integration points each. Each integration point consists of a fiber cross section comprised of aluminum uniaxial bilinear material with F_y of 35 ksi and E of 10,000 ksi. Residual stresses are not considered in this analysis (Bishop, 2017).

3.2 Base Model and Geometric Imperfections

The base model consists of two side-by-side towers each with a top leg extension thirty-two inches long, a bottom leg extension thirty-five inches long, and two five-foot-tall tower frame sections. This geometry was chosen to match conditions measured in the field. The two frame assemblies are tied together using both in-plane and out-of-plane bracing. All section properties are as detailed in Figure 5 and Table 2.

The frame sections are modeled using imperfections consistent with AISC's *Code of Standard Practice for Steel Buildings and Bridges* (AISC 303). An initial out-of-plumbness of $L/500$ was applied over the height of the tower assembly at various locations to be discussed subsequently. An initial out-of-straightness was applied as a half-sine wave with a magnitude of $L/1000$ between the top and bottom leg extension supports.

3.3 Model Geometry and Variables

The following variations on the base model are analyzed:

- Boundary conditions – out of the many possible options for boundary conditions, two were explored in this study as they were likely to bound the true frame behavior: One scenario consisted of pinned boundary conditions at the top and bottom of the tower leg extensions and one consisted of fixed conditions top and bottom. In both cases, the leg extensions were considered continuous at their joint with the tower frame columns and the bracing was considered pinned (moment and torsion released) at its connection to the tower frame columns. The pin boundary condition model is a conservative assumption intended to account for the low moment resistance afforded by the bottom stringer, when present. The fixed boundary condition model represents an upper bound to the capacity based on direct attachment of the tower leg extensions to the hardened slab below and the horizontal formwork shoring above.
- Location of the initial imperfections – the $L/500$ imperfection for out-of-plumbness is varied between three different locations assuming the rest of the joints are collinear: 1) at the top of the upper leg extension, 2) at the intersection of the upper frame column with the upper leg extension, and 3) at the intersection of the lower frame column with the lower leg extension.
- Inclusion of the stringer – models are analyzed both with and without a bottom stringer present.
- Stringer web effective length – since the stringers were modeled as only webs, five variations of the effective length are modeled. The effective lengths range from four inches to sixteen inches, similar to the simplified model discussed above.

3.4 Results

Table 5 tabulates the buckling results from OpenSees when the bottom stringer is not included in the analyses. The results show that for the condition where the ends are pinned, the governing capacity is 11.5 kips per tower column. Conversely, if the ends are fixed to the formwork and the slab below, the capacity of the tower system increases to 34 kips per column tower. In the real structure, the fixity likely lies somewhere between these two extremes. Results for the fixed boundary condition are slightly higher than the ultimate capacity of 24.5 kips provided in the product literature for the five-foot-tall tower shoring system with 39-inch leg extensions.

Table 5: OpenSees buckling results, no stringer

Direction of initial out-of-plumb	**Top and Bottom Conditions**	**Location of initial out-of-plumb**	**Tower capacity per column, P_{cr} (kips)**
Normal to plane of frame	Pinned top and bottom	Tower leg extension	13.7
		Lower frame	13.7
		Upper frame	13.7
	Fixed top and bottom	Tower leg extension	35.3
		Lower frame	34.0
		Upper frame	34.1
In plane of frame	Pinned top and bottom	Tower leg extension	11.5
		Lower frame	11.6
		Upper frame	11.6
	Fixed top and bottom	Tower leg extension	35.4
		Lower frame	34.0
		Upper frame	34.1

Table 6 tabulates the buckling results from OpenSees when the bottom stringer *is included* in the analyses and the system is considered fixed top and bottom (the boundary condition of the stringers to the hardened slab below and the horizontal formwork above). The results indicate the expected behavior of increased buckling capacity with increasing effective stringer length. When comparing these results to those presented for the cases without a bottom stringer in Table 5, the capacity of the system decreases substantially when stringer web buckling is considered. Figure 8 shows the deformed model when the bottom stringer web has buckled.

Table 6: OpenSees buckling results, stringer included

Direction of initial out-of-plumb	Effective length of stringer web (in.)	Location of initial out-of-plumb	Tower capacity per column, P_{cr} (kips)
Outward from plane of frame, fixed top and bottom	4	Upper leg extension	3.4
		Lower frame	3.5
		Upper frame	3.5
	8	Upper leg extension	5.6
		Lower frame	27.4*
		Upper frame	5.6
	12	Upper leg extension	7.8
		Lower frame	7.9
		Upper frame	7.8
	16	Upper leg extension	10.0
		Lower frame	11.4
		Upper frame	10.0

*For this configuration, web buckling of the stringer did not occur.

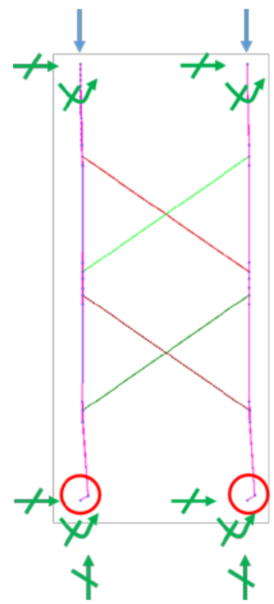

Figure 8: Model with applied loads, boundary conditions, and stringer web buckling indicated

4. Discussion

Stringer web buckling at 44.4 kips vs load-deflection capacity of the tower frame including a stringer of 3.4-10.0 kips

The major difference between these two capacities is that the stringer buckling capacity of 44.4 kips is taken in isolation as compared to the full tower model capacity of 3.4-10 kips.

Another driver for the large difference in capacity has to do with the assumed boundary conditions. When the stringer web is analyzed for buckling, an assumption is being made that both flanges are prevented from lateral translation or rotation while a concentrated load is applied to the top flange. In the case of the tower frame assembly, the stringer that is between the lower leg extension and the hardened slab below is fixed to the slab at the bottom flange. The top flange, however, is not prevented from translation out-of-plane via bracing (no torsional restraint between the two flanges). This creates a condition where the resulting *K*-factor for the system is greater than 0.5 as assumed in the web-only buckling model and can even be much larger than 2.0, leading to a significant decrease in the buckling capacity (Galambos, 1998).

Effect of the 5 foot vs 6 foot tower

Based on the analysis without stringers, substituting the five-foot tall frame sections in lieu of the six-foot tall section only led to a marginal reduction in buckling capacity; 34.5 kips for the six foot tall tower as predicted from the manufacturer down to 34 kips for the five foot tall tower based on OpenSees. It should be noted that OpenSees would have likely predicted a higher capacity than 34.5 kips for the six-foot tall towers, had those cases been analyzed by the authors. The point here is that the overall strength of the five-foot tall frame sections, as predicted by OpenSees, is in the ballpark of the ultimate design capacity reported by the manufacturer for the taller frames.

Effect of stringers

The addition of stringers had the most dramatic effect. If stringers are considered in the analysis below the bottom leg extensions, a significant reduction in capacity is predicted to occur. The capacity decreases from 34 kips for the case without stringers to between 3.4-10.0 kips for the cases including a bottom stringer (where the specific resulting capacity depends on what length of stinger web is considered to participate). Consequently, the effect of modeling the stringers is on the order of the result predicted in OpenSees by modeling the frame assembly (without stringers) as a pinned end (11.5 kips). Contractors should not use sections of stringers with unbraced webs to extend the shoring height.

Effect of various boundary conditions

Another comparison of note is the difference between the capacities as predicted by OpenSees of the five-foot tall tower frames (no stringers) with pin-pin versus fix-fix boundary conditions (11.5 kip capacity vs 34 kip capacity, respectively). The fixed capacity is well above the unfactored design load while the pinned capacity is just below. Based on our observations in the field, shoring designers should not be assuming fixed base boundary conditions without justification. For unfactored design loads closer to the pinned-ended capacity, the effect of five-foot versus six-foot tower height may be significant given that a capacity of 11.5 kips for the five foot tower frames is so close to the unfactored design load.

Disclaimer

The views expressed herein are solely those of the authors and do not necessarily represent the position(s) of Exponent Inc. or any other individuals.

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