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Load Tests of Common Shoring Towers: Typical Detailing and Resulting Capacity Reduction

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Abstract
Following an in service collapse, three shoring towers were tested to failure at the University of Texas at Austin's Ferguson Structural Engineering Laboratory (FSEL) to better understand the cause of the collapse. Both numerical analyses and experimental results indicated a stability limit state governed the load capacity that may have not be adequately considered in the design of the shoring towers.

The shoring system tested is widely used in construction and is constructed of modular aluminum components. A typical four-leg tower is constructed with paired frame segments, each containing two column legs. Frame segments are stacked and fitted with adjustable height extensions. Tower legs support a system of cribbage that includes beams and girders, which in turn support wooden formwork.

The detail of placing a beam or girder directly over a column is known to increase the effective length of the supporting column, thereby reducing its buckling capacity. A number of structural collapses over many years have been attributed to this destabilizing detail. Consequently, the detail is typically either avoided or is modified to minimize its destabilizing effects. The shoring system tested includes the destabilizing beam-over-column detail without modification. To investigate the effects of the detail on the capacity of a shore tower, tests performed at FSEL included specimens with and without the beam-over-column detail.

The first test, of three, applied load directly to the legs of a four-leg tower, and did not include the destabilizing beam-over-column detail. A buckling failure occurred at 96% of the manufacturers' provided ultimate load. The second and third tests included the beam-over-column detail and were loaded through wooden formwork and a cribbage system, to be representative of typical field conditions. The results of these tests, which included the detail discussed, showed an approximate reduction in ultimate load of 40% prior to failure.

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1. Introduction

In March 2014 during the placement of an elevated concrete slab, a collapse of the supporting shoring occurred. Fortunately no workers were seriously injured, although five men rode the debris to the lower level. Photographs of the collapse are shown in fig. 1. The ensuing investigation led to analyses and eventually experimental testing which was able to show that the stated capacity of the shoring system was overestimated as a result of a stability limit state not adequately considered in its design.

Figure 1: Photographs of Collapse, Ariel View and Below Level of Collapsed Shoring

Although the collapsed components were specific to a single shoring manufacturer, the construction details that led, in part, to the collapse are common to several shoring systems. The popularity of these systems is due in part to their lightweight aluminum construction, which reduces labor costs, and the modularity that is offered by the system. Horizontal components are available in multiple lengths, and vertical components feature adjustable heights allowing the system to accommodate variable configurations. Thus contractors are able to limit the amount of supply required to prepare several placements. Typically these systems are rented from a manufacturer that also supplies erection drawings and design calculations.

The collapse that occurred in March 2014 was the result of the shoring system manufacturer not adequately considering stability effects of their system, but was also a result of calculation errors that did not consider load effects of continuous beams, and neglected a significant amount of dead load at the location of the collapse. Where the collapse occurred, the design detailed shoring towers that were at the allowable limit of the frame legs, and were ultimately not constructed as designed. Consequently, the collapse occurred due to an agglomeration of several factors. If any one of these factors had been absent it is likely that the collapse would not have occurred, and as a result the review of the shoring system including the tests presented herein would also not have occurred.

2. Shoring System and Components

2.1 Description of Common Shoring System

The system is constructed of aluminum shoring towers and aluminum cribbage consisting of horizontal members commonly referred to as 'stringers' and 'beams'. To support an elevated concrete slab or beam, plywood formwork is connected to the top flanges of the horizontal members referred to as 'beams'. The beams span between supporting horizontal members referred to as 'stringers'. The stringers are in turn supported by an aluminum tower typically consisting of four legs that act as columns. Both stringer and beam members are I shaped sections that are made to be joined by bolts, clips or connected to formwork by nails. A typical configuration of the shoring system is shown in fig. 2. Also included in the figure is a cross sectional view of both beam and stringer members. Note that stringer sections are typically deeper than beams. The stringer members typically feature enlarged flange tips that conveniently allow connections by specially designed bolted clips.

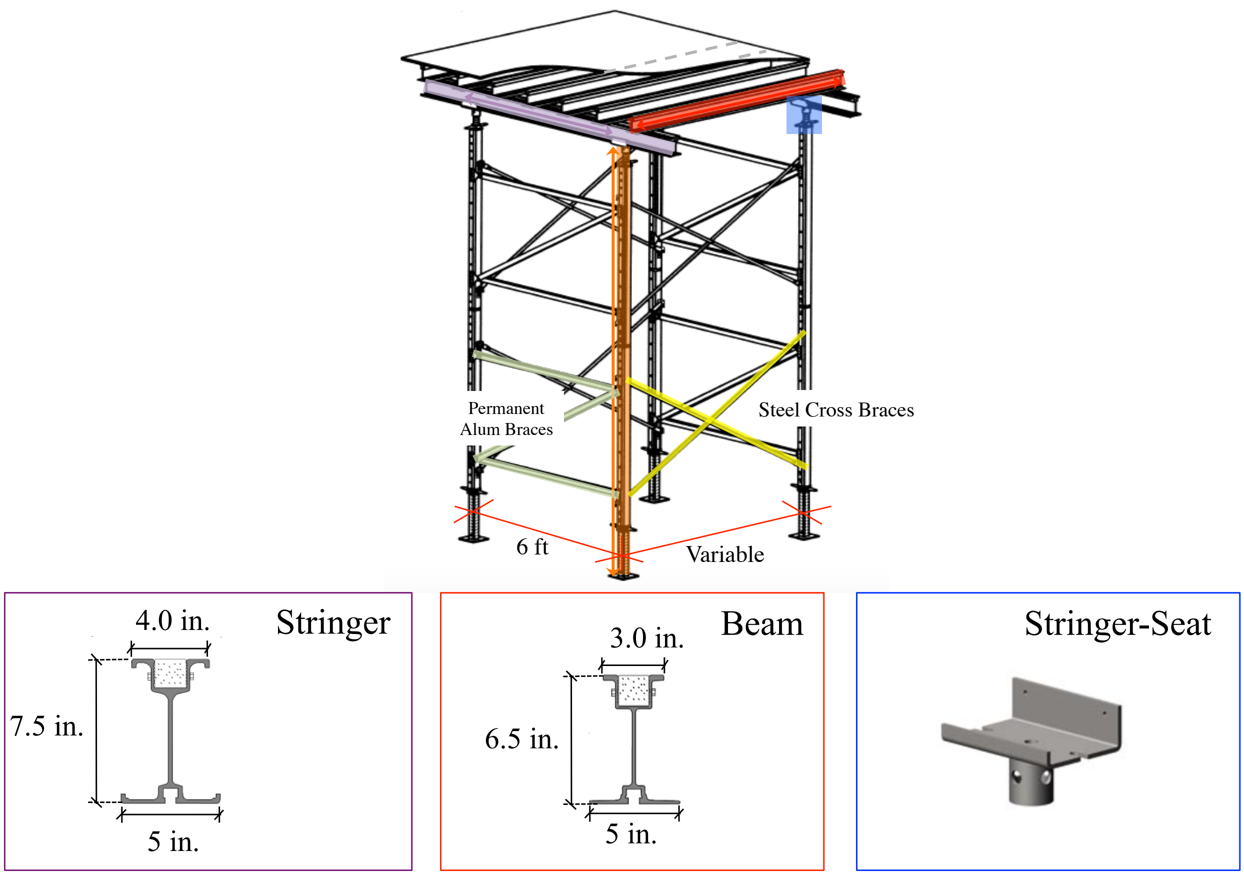

Figure 2: Typical Shoring Tower, Cribbage and Formwork, Isometric View

The frame legs making up a tower are provided in sections containing a pair of legs connected by a system of aluminum tube braces. An example of a frame section is shown in fig. 3. The frame legs are fit with adjustable height extensions that are placed at both tops and bottoms of the legs. The leg extensions are in turn connected to either a base plate or stringer-seat depending on location. The extensions are smaller in cross section than the frame tubes and are connected by sliding the smaller extensions into the larger frame legs.

Figure 3: Common Frame Section

Baseplates typically sit on concrete slabs or mudsills without a positive connection. Alternatively, stringer-seats may be positively connected to stringer members that they support via a small bolt that fits into the center of the stringer's bottom flange.

2.2. Beam-over-Column Detail of Interest

The condition of the beam over top of column has been the subject of past work and has been observed to result in stability failures (Hauck and Moe, 2010). This condition typically occurs in steel construction where continuous beams exist and beam continuity is provided by extending the beam length over the column.

The stability issue arises from the development of second order effects within the web of the beam member. Second order effects within the web result in the propensity of the bottom flange of the beam to translate and rotate when the beam is subject to vertical load (fig. 4). The AISC Manual of Steel Construction (AISC 2015) provides a discussion on the beam-over-column detail, and provides recommendations to mitigate the destabilizing effects of this detail. These recommendations include the addition of a stiffener in the beam web above the column (fig. 4), or the addition of bracing at the beam bottom flange.

In the case of the shoring assembly, webs are unstiffened, and the bottom flanges of horizontal members are typically unbraced, with the exception being any bracing provided by the member's support. Given the variety of configurations that horizontal shoring members may see in their service life it is understandable that a permanent stiffener is not employed. In the case of the stringers, support is provided by the variable length leg extensions. Therefore, the supporting extensions must provide any resistance to the resultant forces at the stringer's bottom flange. Moreover, the available resistance, or effective bracing provided by the support condition degrades with increased height of the extension (fig. 4). As such, the extensions placed at increased heights are more susceptible to the unstable condition. As will be described in the subsequent section, such affects were not considered in the determination of the capacity of the frame legs and extensions.

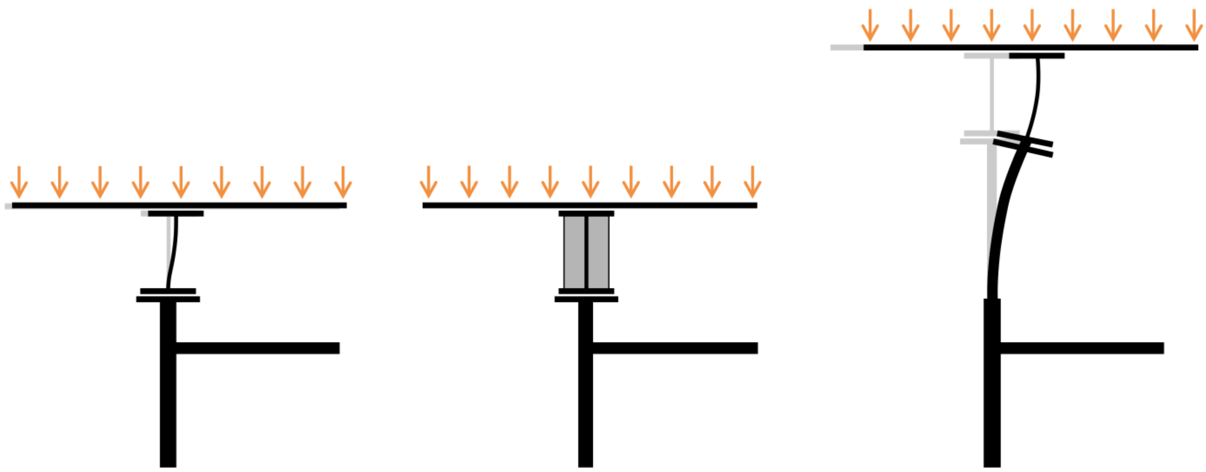

Figure 4: Beam Column Instability

3. Reported Capacities and Typical Design

Shoring systems are typically designed on a member-by-member basis with demands calculated by tributary area. The calculated member demand is often compared to design guides and tables that are entered with a member type and span configuration. The design guides are developed with simple structural analyses with limit states of shear, moment, reaction, or deflection of the horizontal members, and axial load of vertical members. The output of these guides is typically provided as an allowable uniform load, which can be compared to demand. This type of design is efficient and is easily modified in the event that an allowable load is exceeded.

The determination of member capacities is determined by testing in accordance with recommendations by the Scaffolding, Shoring and Formwork Institute (AASE, 2013). Recommendations for test requirements are provided, however these recommendations are limited and allow for interpretation by test engineers (SSFI, 2007). Ultimate values determined in testing are divided by a factor of safety established by ANSI/ASSE A10.9-2013 "Safety Requirements for Concrete and Masonry Work" (AASE, 2013). Factors of safety are set at 2.0 for horizontal members, and 2.5 for frame towers.

3.1 Frame Tower Capacity

The allowable load capacity of shoring frame legs is typically a function of the adjustable height extensions placed at either end of the frame legs. The capacity of a four-leg tower, the typical configuration, is determined via testing of the frames. Frame legs are fit with extensions set to a length/height of interest, and increasing axial load is applied to the setup until failure of the frame occurs. The ultimate load, determined in the test, is averaged over a minimum of three tests and divided by the required factor of safety to determine and allowable load at the tested extension height (SSFI, 2007, ASSE, 2013). Guidance for these tests notes that linear interpolation may be used between averaged test results of different extension heights to determine the frame capacity at extensions not tested. Design engineers use the plot, or series of linear functions, resulting from these analyses to develop erection plans and supporting calculations.

Documents reviewed during the collapse investigation provided insight into how the tests were completed. In most cases, tests were completed with extensions set to equal lengths at the tops and bottoms of the frame legs. Both top and bottom extensions were fit with a base plate to facilitate bearing and load transfer, and load was applied via what test reports described as a floating strongback. The floating strongback is assumed to be a member sufficiently stiff to distribute load equally to the four-leg test specimen. Hollow core rams were used to pull four steel rods distributed about the strongback, which in turn applied compressive load to the frame. Applied load was tracked via a pressure gauge, the reading of which could be converted to force with the area of the rams employed. As noted on the test documentation the administrators of the test assumed that the configuration employed was free to translate laterally, and thus the test was stated to be representative of a sway condition.

The assumption of a sway condition may not be completely consistent with the actual test conditions. The tie rods used to transmit axial load provide some lateral resistance. Any lateral translation of the top of the test frame is resisted by the developed lateral component of the tensile force in the tie rods, fig. 5. The lateral resistance is proportional to both the magnitude of tensile force in the rod and the rotation of the rod.

It should also be noted that the sway condition assumed might also be prevented in practice. Typical concrete construction casts vertical members such as shear walls and columns prior to the elevated concrete slabs or slab and beam systems. Vertical members are also used to support formwork to prevent cantilevered conditions within shoring members. This support can provide a path for lateral load transferred by a plywood diagraph that is also connected to the shoring system.

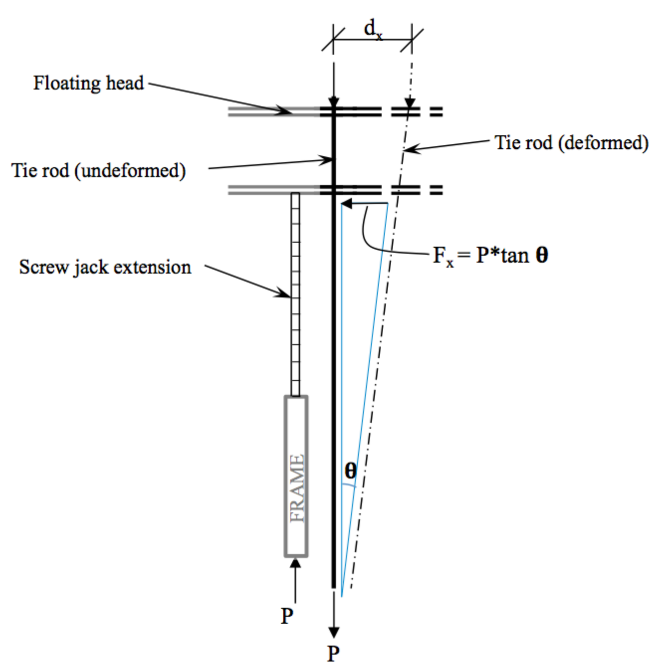

Figure 5: Tie Rod Resistance to Lateral Translation

Base plates in similar systems develop some rotational fixity (Eröz et al. 2008). In practice base plates used vary in size and thickness. Employing the smallest base plates in terms of plan and thickness dimensions could provide conservative results in testing however no indication was provided that this was considered. Base plates provided in practice vary in plan dimensions and are often rectangular such that column capacity could vary depending on plate orientation.

The most significant observation of the test results is that the beams and stringer cribbage is noticeably absent. These members are tested, or analyzed separately. As such the beam-over column connection detail is not present in the tests.

3.2 Capacity of Horizontal Members

The design of horizontal members is governed by one of four limit states: deflection, bending, shear, or compression at a point load reaction. The capacity of the horizontal members, either beams or stingers, with respect to the individual limit state is required to be determined by testing (SSFI, 2007).

Manufacturer's documents available from the collapsed system indicated that testing was used to determine the magnitude of a reaction force that the elements could resist. This particular system allows the use of a butt joint of stringer members over a stringer-seat. This joint detail was determined by the manufacturer to govern the reaction capacity of stringers and is of particular interest to instability of the detail. The test setup used to determine the capacity of the stringer included two simply supported stringers butt jointed over a center support, as illustrated in fig. 6. Equal load was applied to the center of each span. Load was increased until deformation, described as twisting or buckling, of a web at the center support was observed. The test results differ from typical in-place conditions due to the stiffness of the supports used. The tests did use a typical stringer-seat onto which the stringer was placed. However this seat was connected to a short steel tube that was in turn rigidly connected to a larger test frame support. As such, the support conditions provided by the test were significantly stiffer than those possible with long leg extensions.

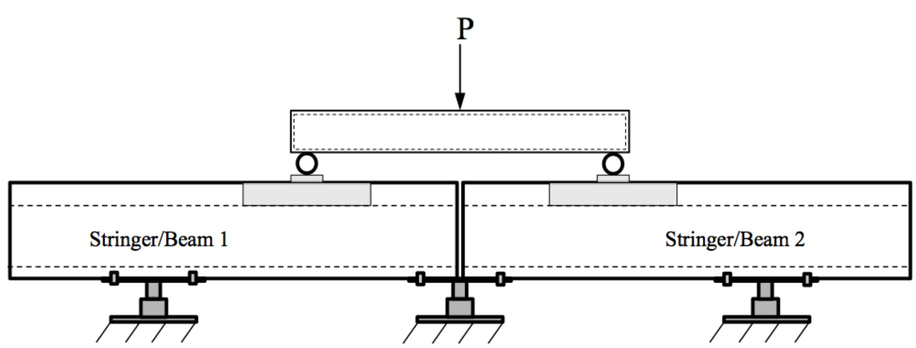

Figure 6: Test Setup, Determination of Stringer Reaction Capacity

Tests to determine member capacities with respect to the remaining limit states were not available to review. However, submitted design calculations show the use of section and material yield strength used to determine member bending capacity and deflections. Similarly, the authors were able to use section and material properties to estimate member shear capacity. Estimated values compared well to magnitudes given within the manufacturer's documents.

4. Experimental Evaluation

Three tests of the shoring system were completed at the Ferguson Structural Engineering Laboratory (FESL) at the University of Texas. The results of these tests were used 1) to compare the capacity of frame members to the manufacturer provided axial load capacities, and 2) to show possible effects of the inclusion of cribbage above the frame legs on the system capacity, and more specifically, the effects of the beam (stringer) -over-column condition. As a result of the typical design methodology employed, the system capacity is presumed to be a function of individual member capacities. As such, the specimens tested were setup such that the axial leg load of the frame elements would have governed the design.

Each of the three tests was based on a four-leg frame system. The legs were spaced at 6 and 8 ft in plan dimensions representing common plan dimensions as determined by review of provided construction documents. The frame members tested consisted of permanent aluminum bracing in one plane, spacing legs in this plane at a constant 6 ft. In the orthogonal direction 1 in. diameter steel tube cross braces were connected to frame legs. The cross braces are available in various lengths to accommodate leg spacing in this direction between 4 and 10 ft. Each of the brace types and dimensions are shown in fig. 2.

The four-leg frame systems were erected by assembling vertically the extension legs on top and bottom of the aluminum frames. During the erection, the four legs of each tower were carefully aligned and kept plumbed so that excessive initial imperfections were avoided. After the full erection of the three towers, initial imperfections of the four legs were measured and a maximum out-of-plumbness of approximate H/500 was recorded (H=tower height).

Axial load was applied to the respective test specimens through a set of four steel rods tensioned with hollow core hydraulic rams. Load cells were placed in line with each of the four rams and loads recorded through the duration of all tests. In each of the three tests, leg extensions were set to the maximum allowable length, 39 inches. In each test, leg extensions and frame legs were instrumented with string potentiometers to monitor lateral displacement in the orthogonal axes of the frame, and with uniaxial strain gages to confirm an even load distribution among the four columns. Additional string potentiometers were employed to measure vertical displacement.

Tie rods were placed through double channel members spanning the 6 ft frame direction. In all tests lateral translation of the test specimen was restricted at the top of the tested frame. This restriction was carefully applied such that deformation of the frame leg extensions was not prohibited.

4.1 Test No. 1, Comparison to Manufacturer Testing

The first test was performed on a frame system that included only the frame legs, extensions at the top and bottom of the frame legs, bearing plates at the floor, and stringer-seats at the top extensions. The frame components used in the test consisted of a single pair of 6 ft frame components. Thus, the total height of the test specimen was approximately 12.5 ft (150 in.). Load was applied to the first test through a stiff steel frame constructed of HSS members sized to sit in each of the four stringer-seats placed atop the top extensions (ref fig. 8). Consequently, there was no destabilizing beam-over-column detail represented in this specimen, due to the large torsional stiffness of the HSS members. The specimen of this test was configured to replicate that of the reported tests used to develop the published capacities of the frame members.

The test was concluded with an elastic buckling failure of the frame. The failure occurred at a total applied load of approximately 93.2 kips, or 23.3 kips per leg. The failure occurred with two frame legs displacing in the direction of the permanent aluminum bracing. Recorded load versus mid-height frame leg lateral displacement is plotted in fig. 7. The displacement record shown in the figure was selected from a mid-height string potentiometer located in the plane of the buckling direction on one of the two failed frame legs. As anticipated, the load versus displacement behavior shows little lateral displacement (0.08 in.) up to the point of buckling. The magnitude of load shown in this figure is one quarter of the total applied load.

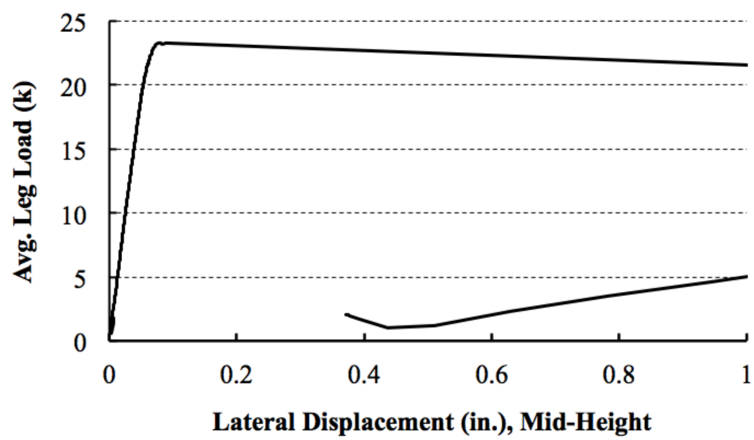

Figure 7: Test No. 1, Average Leg Load versus Displacement

The guide used to design the frames noted an allowable load of 9.7 kips per leg where extensions are set to 39 inches. Multiplying the allowable value by the stated factor of safety, 2.5, provides an ultimate leg load of 24.3 kips per leg. The observed failure in test 1 occurred at an applied load of 96% of the purported ultimate leg load.

For comparison, the elastic critical load of the frame assuming a uniform cross section equal to the frame moment of inertia, and assuming pined end conditions results in a predicted failure load of 13.5-14.4 kips per leg. In comparison to the calculated pinned end critical load, the measured failure load in the test may reflect the contribution of the rotational stiffness of the base plates at the bottom of the column extensions, and possibly the rotational stiffness of the stringer-seats and HSS members at the top of the column extensions.

Figure 8: Load Application, Test No. 1 Left, Test No. 2, Right

4.2 Test No. 2, Inclusion of Horizontal Members

The second test was performed with the same frame dimensions as the first test. The test was modified to include a system of cribbage of beams, stringers, and 3/4 in. plywood, with concrete slabs placed over the plywood. Photos in fig. 7 show the top of the specimens, and the resulting load path differences between Test No. 1 and Test No. 2.

Two stringers were placed at either side of the frame, parallel to the frame's permanent bracing. The stringers were 8 ft in length and were centered over the frame legs below. In the perpendicular direction a total of 10 beams were placed over the stringers. The beams, each 10 ft in length, were centered over supporting stringers. Connections between stringers and stringer-seats, and beams and stringers were completed in the most conservative interpretation of the manufacture provided recommendations. A precast concrete plank, approximately equal to the plan dimensions of the frame tower and 3 in. thick was placed onto the plywood formwork to distribute applied load. Prior to its placement, the plank was cut parallel to the stringers orientation such that the stiffness of the plank did not affect frame deformation perpendicular to the stringers, the predicted direction of failure. The test employed the same tie rod, double channel, hydraulic ram loading system use in the first test.

The configuration of the test, specifically cribbage spacing, was designed such that the axial load capacity of the frame legs was the governing strength limit state.

Applied load resulted in lateral displacement of each of the four frame legs of the test specimen. Each of the legs deformed in accordance with the first bucking mode with mid-height lateral displacements being in the plane of the 8 ft plan dimension, or the direction of the steel cross braces. Deformation of each leg was in the same direction.

The maximum load achieved in the test was approximately 60 kips, or 15 kips per leg. At the point of maximum load the corresponding maximum mid-height displacement was approximately 1.4 inches. The load versus lateral displacement behavior of this test is presented in fig. 9. For comparison the data shown in fig. 7 (Test No. 1) is also provided. Note that in Fig. 6 data recorded following the buckling failure is not presented for clarity.

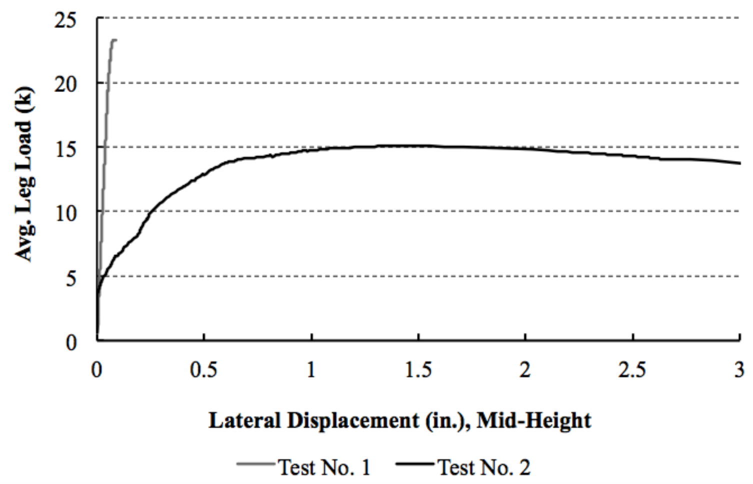

Figure 9: Test No. 2 (Black), Average Leg Load versus Displacement

The results of the test show a change in stiffness as a result of the included cribbage as well as a difference in the maximum applied load. The maximum load applied in the second test, 15.1 kips per leg is approximately 65% of the maximum 23.3 kip per leg load observed in the first test.

4.3 Test No. 3, Two Stacked Frame Members

The third test was performed with the sole modification from Test No. 2 being the inclusion of a second tier of frame members. The frame members were stacked to increase the height of the test specimen to a total height of approximately 18.5 ft (222 in.). The published capacity of the frame sections indicated that frame members may be stacked. The capacity chart noted that stacks beyond two frames require bracing to develop the capacity stated given by the chart. As such the test employed no additional bracing to the frame.

Similar to Test No. 2, the frame legs displaced in the direction of the steel cross braces with applied load, and the direction of the displacement was the same in each of the four legs of the specimen and the same as that observed in Test No. 2.

The load versus lateral displacement behavior of the third test is shown in fig. 10. The results of Test No. 1 and Test No. 2 are also shown in the figure. Within the first 1.5 in. of lateral displacement the maximum load applied was approximately 13.4 kips per leg. Beyond this displacement the test reached a maximum applied test load of approximately 13.9 kips per leg at a displacement of 2.9 inches. As seen in the fig. 10, the applied load is nearly constant with increased lateral displacement in the range of 1.5 – 1.8 inches. At a displacement of approximately 1.9 in. load began to increase slightly again. This additional post-peak load capacity occurs at very large lateral displacements, and is therefore not considered indicative of the practical load capacity in the field, as displacements of this magnitude would likely be prohibited in practice.

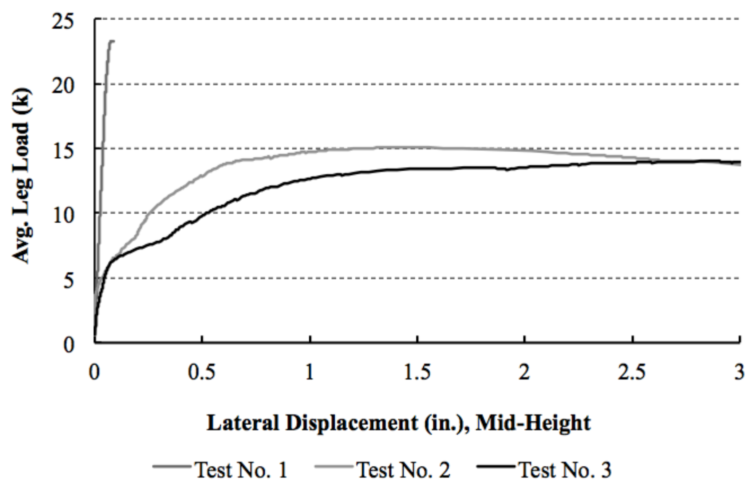

Figure 10: Test No. 3 (Black), Average Leg Load versus Displacement

The results of the test do show a noticeable decrease in the load capacity of the system as a result of the inclusion of a second set of frame members. In comparison to Test No. 1, the maximum Test No. 3 load of 13.4 kips per leg is 58% of the maximum Test No. 1 load and 55% of the ultimate load given by the design chart. The test shows that the increased height results in a decrease of the system capacity, approximately 11% in comparison to the second test. It should be noted that documents reviewed did indicate that frame towers constructed with stacked frame elements were tested in the development of the frame capacity chart.

5. Conclusions

A collapse of shoring used to support an elevated concrete slab and beam system led to a review of system capacities that included a review of documentation supporting the capacity of system elements, numerical analysis (not discussed herein), and experimental tests of the shoring system in three separate configurations.

The review showed that the system, as used in the field, is erected with a beam-over column connection detail. This type of connection type has been shown to result in stability failures, if not properly detailed with web stiffeners or additional out-of-plane bracing at the top of the column. Neither of these mitigating features were incorporated into the design or construction of the shoring system reviewed.

The review also showed that the capacity of the system was determined by the manufacturer, in part, by experimental testing. However, the tests performed to determine the capacity of the vertical elements did not include horizontal members and thus the tests did not include the destabilizing effect of the beam-over-column detail.

To determine the system capacity with the inclusion of the beam-over-column detail three full-scale tests were performed at the University of Texas' structural engineering laboratory. The first test was completed in a manner similar to that of the test used by the manufacturer to determine frame capacities. The remaining two tests included the cribbage that results in the beam-over-column detail and is inherent to the system's method of formwork support.

The first test resulted in a failure mode and load similar to that observed in the tests performed by the manufacturer. The test frame tower failed at a load of 95% of that reported by the manufacturer as the average test results at the same specimen height.

The second test was modified from the first solely by inclusion of a system of cribbage that included horizontal members and plywood formwork. As stated, these members are required in the shoring system in practice to support formwork. The cribbage members were placed such that the critical limit state by of the typical design method was the axial load of the frame. The result of this test was a 38% reduction from the purported ultimate load, and lateral displacements of the frame legs significantly larger than those observed in the first test.

The third test, which included a second vertical tier of frame members to increase the height of the test tower showed a further reduction in load capacity, again with large lateral displacements. In this test a 45% reduction from the purported capacity was observed.

The experimental results, summarized in Table 1, show that the inclusion of the beam-over-column detail, as used in the actual field application of this system results in a significantly reduced load capacity.

Table 1: Summary of Experimental Results

Specimen	Cribbage Included	Frame Ht. (ft)	$P_{Failure}$ (k)	Percent of P_0 (%)
Manufacturer	No	12.5	24.3	-
Test 1	No	12.5	23.3	96
Test 2	Yes	12.5	15.1	62
Test 3	Yes	18.5	13.4	55

While a single shoring system was tested in this work, the detail observed is common in the industry. As discussed, the industry required testing used to determine the capacity of these systems, which is in turn used in design, does not incorporate the problematic beam-over-column detail into tests. As such, the shoring systems are being used in practice with an overestimated level of confidence based on assumed factors of safety and system behavior.

Acknowledgments

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