Investigation of the September 6, 2012 Partial Collapse of a Slab during Construction at Hyatt Place, Omaha, NE

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1. Introduction

On September 6, 2012 at approximately 5:00 a.m., a partial collapse of the second level slab occurred during construction of the Hyatt Place Hotel in Omaha, NE. At the time of the collapse, the northwest section of the second level was being placed with fresh concrete over the formwork. Twenty-five employees were working with the wet concrete. Six of them on the formwork fell 10 to 18 feet below to the ground level. Three employees were injured.

The Occupational Safety and Health Administration's (OSHA) Region VII Administrator requested the Directorate of Construction (DOC), OSHA National Office, to provide engineering assistance for a causal determination. A structural engineer from DOC arrived at the construction site and examined the failed shoring system on September 11 and 12, 2012. During this trip, the engineer also interviewed construction workers and other professionals at the construction site, took photographs, obtained construction documents, and examined failed structural elements for analysis later.

OSHA's Omaha Area Office provided the following documents required for the analyses:

- Architectural drawing A1.2, entitled Level 2 Floor Plan.
- Structural drawings, S0.1, S1.0, S1.1, and S1.2; entitled Structural Notes; Auger Cast Pile Plan; Foundation Framing Plan, Level 1, Lobby; and Framing Plan, Level 2, Garage; respectively.
- Contract specification, Section 033000 -- Cast-in-Place Concrete.
- Preliminary shoring drawings (marked "not for construction"), prepared by Ischebeck, dated June 12 and July 3, 2012.
- Catalogs of concrete forming and supporting systems, published by Ischebeck, 2009.
- Load test results of the HV Maxi Legs with photographs, performed at Lehigh University, by R. H. Tulis, PE, October 25-26, 2005.
- Soil compaction test report of May 2, 2012, prepared by Olsson Associates.
- In-place field density tests of June 14, 2012, performed by Olsson Associates.

- Analysis of the September 6, 2012 shoring collapse, prepared by Ischebeck, dated September 11, 2012.
- Engineering properties of the HV Maxi legs (proprietary information), provided by Ischebeck, 2013.
- Post-incident final shoring design, approved by Jeffrey W. Shaffer, Shaffer & Stevens, PC, October 2012.

2. The Project

The project consisted of construction of a 159-room, ten-story building, Hyatt Place Hotel with self-contained parking at the intersection of Jackson Street and 12th Street, Omaha, NE. The following are the key participant in the project.

Owner:	Hyatt Hotels Corporation.
Developer for Hyatt affiliates:	Woodbine Development Corporation.
Architect, Engineers & Planners:	DLR Group Inc., a Nebraska Corporation.
General Contractor:	Hawkins Construction Company of Omaha, NE.
Shoring Distributor:	Jobsite Supply of Indianapolis, IN.
Shoring Manufacturer and Preliminary Shoring Designer:	Ischebeck USA Inc. of Naples, FL.
Post-incident Final Shoring Designer:	Shaffer & Stevens, PC, of Omaha, NE.
Shoring Erector:	Hawkins Construction Company of Omaha, NE.
Concrete Subcontractor:	Innovative Concrete Inc. of Omaha, NE.
Soil Testing Agency:	Olson Associates of La Vista, NE.

The building was approximately 130' by 190'. The first five levels consisted of a parking facility, and the remaining levels were for the hotel rooms and its amenities. The construction of the project started in April 2012. The construction of the parking garage levels consisted of 9" thick post-tensioned concrete slab. The slab at first floor level was on grade and the garage ramp was on a high density Styrofoam. However, the ramp was not yet constructed as planned at the time of the incident.

Ischebeck USA Inc., (Ischebeck), prepared the preliminary design of the formwork for casting concrete at the parking garage levels using their self-manufactured shoring components except for the top panel/sheeting. Ischebeck issued preliminary formwork drawings for the entire parking area including miscellaneous sections, on June 12, 2012 and July 3, 2012. The formwork drawings were for information purposes only, and were clearly marked “not for construction”. In addition, these drawings were neither signed, stamped by Ischebech; nor approved by the structural engineer of record.

The shoring for the formwork system consisted of HV Standard Legs, HV Maxi Legs, Megashore Legs and Ledger Frames (Gates) on a 6' x 6' grid. The post shores were supported on 2x12 mudsills. The mudsills were placed over 4" thick crushed rocks on compacted subgrade. On top of each post shore a drop-head was provided to support the main beams V in the north-south direction. The main beams V were supporting the secondary beams H spaced approximately 18" on center in the east-west direction. The ¾" thick plywood was used on top of the primary and secondary beams. HV Maxi Legs in two tiers were used where the shoring height exceeded approximately 19'-2". Ledger Frames (Gates) were used either to brace the post shores at the cantilevered beam area or at the sloped base grade area.

The contractor erected the formwork for the second level using the preliminary shoring drawings of June 12, 2012 and July 3, 2012. The contractor finished placing wet concrete at the second level of the parking garage for an approximate area of 18,000 ft^2, i.e., ¾ of the entire second level in the east portion of the building. The concrete for the slab was placed by the pumping method. The volume of concrete pumped was approximately 500 yd^3 with a total weight of approximately 2 million pounds. The placement of concrete was completed on August 28, 2012, and no problem was encountered. The incident occurred on the next pour of concrete described below.

3. The Incident

The area where the post shores, formwork and slab failed was near the northwest corner of the building which was bounded by grid lines 1.7 to 3 in the east-west direction and by the length of the concrete advance in the north-south direction, see figure below. For the second level slab, the contractor erected the post shores and formwork using the Ischebeck's preliminary shoring drawings issued on June 12, 2012 and July 3, 2012 (See Figures 1 and 2) with some modification at the cantilevered slab on the north and the west end of the building. The modification for the shoring layout is shown in Figures 3 and is discussed later.

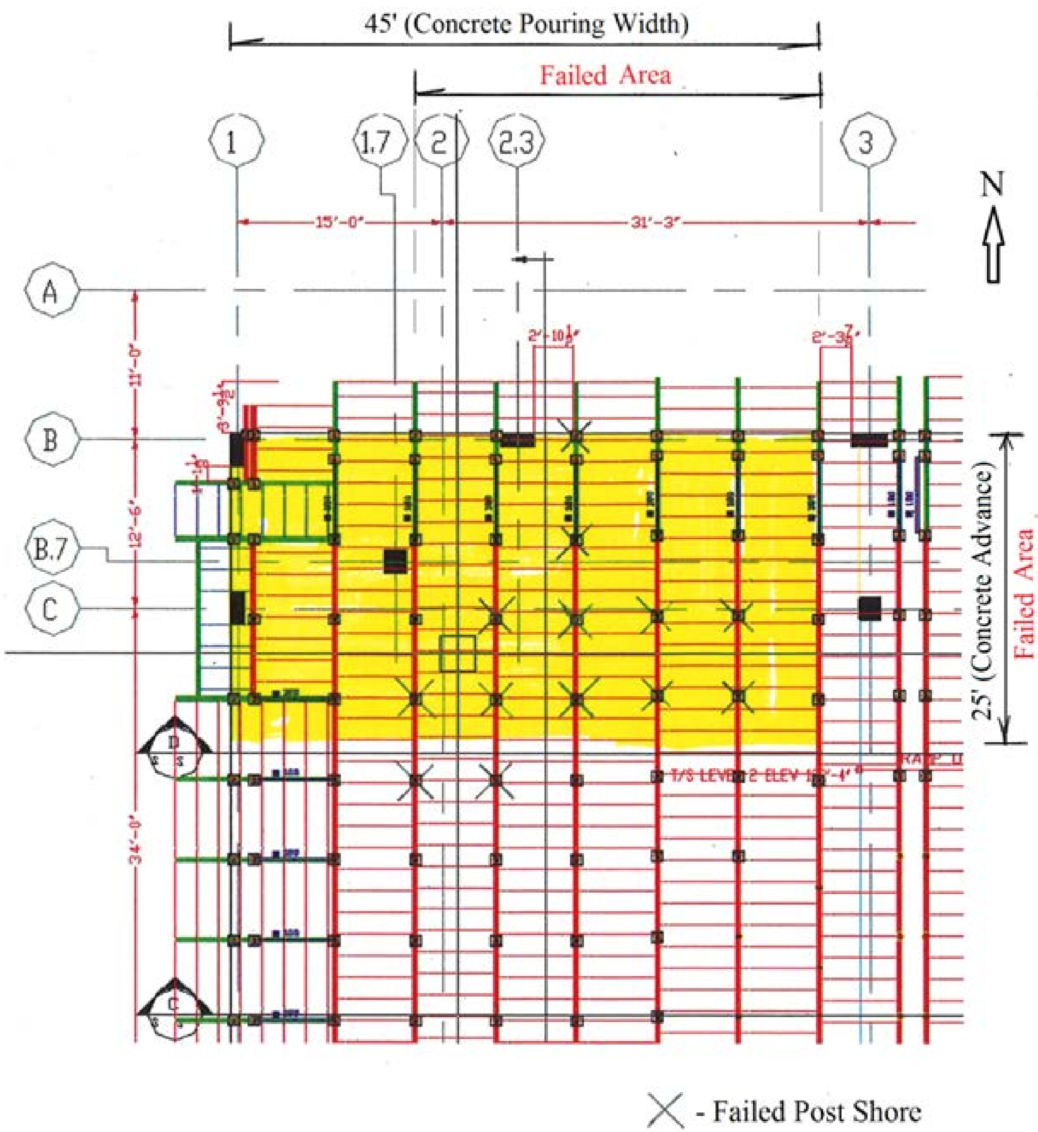

Figure 1. Plan view of the collapsed area (Modified from Ischebeck, 2012).
Note that the yellow highlighted area was the limit of wet concrete placed at the time of the incident.

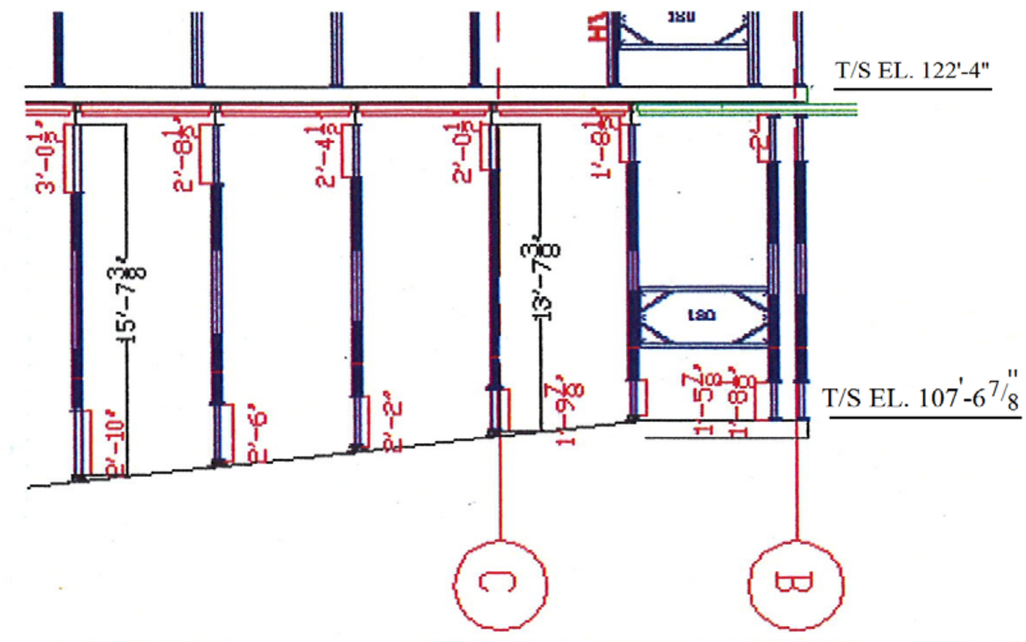

Figure 2. Elevation of the collapsed area per preliminary design (Modified from Ischebeck, 2012).

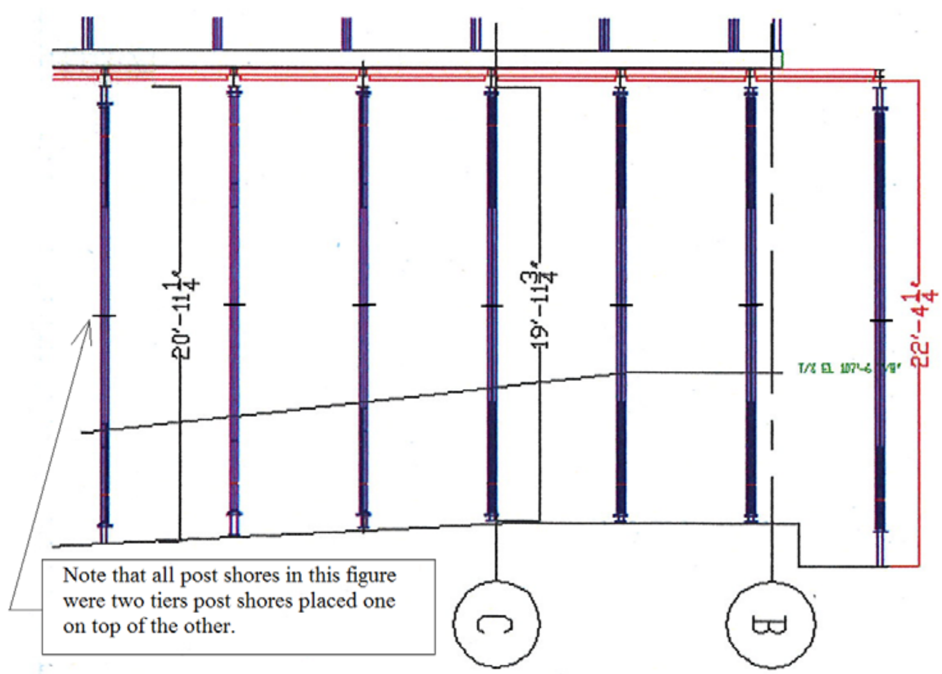

Figure 3. Elevation of the collapsed area as installed (Modified from Ischebeck, 2012).

Over the modified shoring, the contractor began to place wet concrete by pumping from the north end toward the south with the slab width of about 45'. After the leading edge of the concrete advanced to 25', and after pumping about 32 yd^3 of concrete, the entire formwork, including the slab, collapsed. The six employees working on the second level during concrete pumping fell 10 to 18 feet to the ground level below and three of them were injured. Figures 4 through 9 present the condition of the slab and formwork after the collapse.

Figure 4. Collapsed shoring (Looking from the ground level toward the north).

Figure 5. Collapsed shoring (Looking from the ground level toward the south).

Figure 6. Collapsed concrete forming at the second level (Looking toward the northwest).

Figure 7. Collapsed concrete forming at the second level (Looking toward the north).

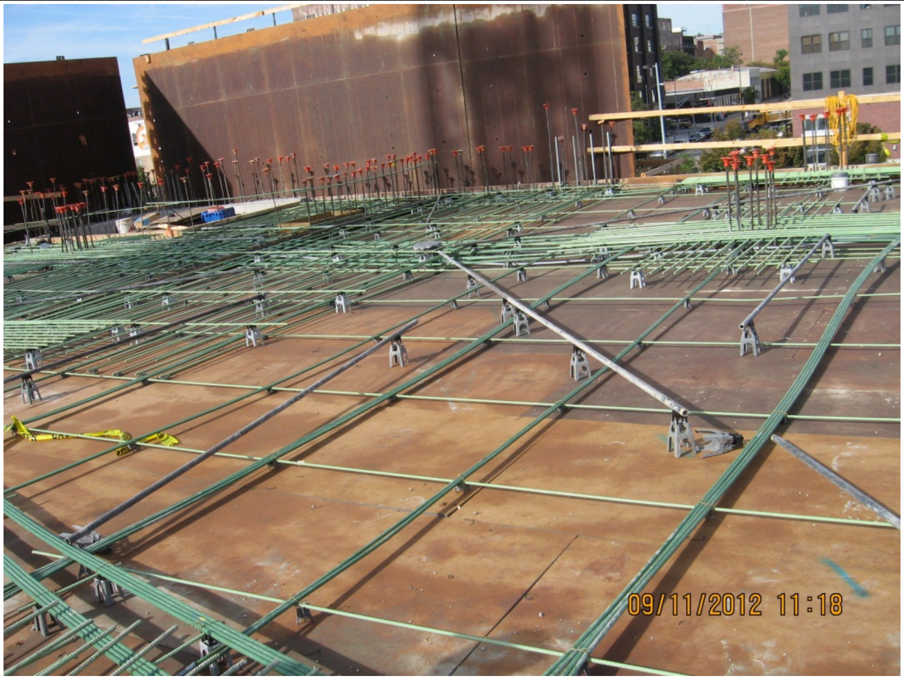

Figure 8. Concrete forming and reinforcement at the second level (Looking toward the southwest).

Figure 9. Concrete forming and reinforcement at the second level (Looking toward the south).

4. Description of the Collapse

Field Observations

During the September 11-12, 2012 visit to the incident site, we observed the following:

1. The preliminary shoring drawing Sheet No. 5 (Figure 2) indicated that at the collapsed area, the post shores below the second level were to be supported on a structural slab at an elevation of 107'-$6^{7}/_{8}$". However, the structural slab was not installed at the time the shoring was erected. As a result, the post shores in this area were installed from the grade level up and the shoring height increased from approximately 13'-6" to an average of 20'-1". Therefore, the contractor used two tiers of HV Maxi Leg in this area, one placed over the top of the other (Figure 3).
2. The contractor supported the two-tier post shore (HV Maxi Leg) on a single 2x12 timber plank (mudsill) on 4" thick crushed rock, a deviation from the preliminary design. The base plate of the post shore was nailed only at two places on mudsill. The adjoining mudsills were not connected to each other (Figures 10 and 11).

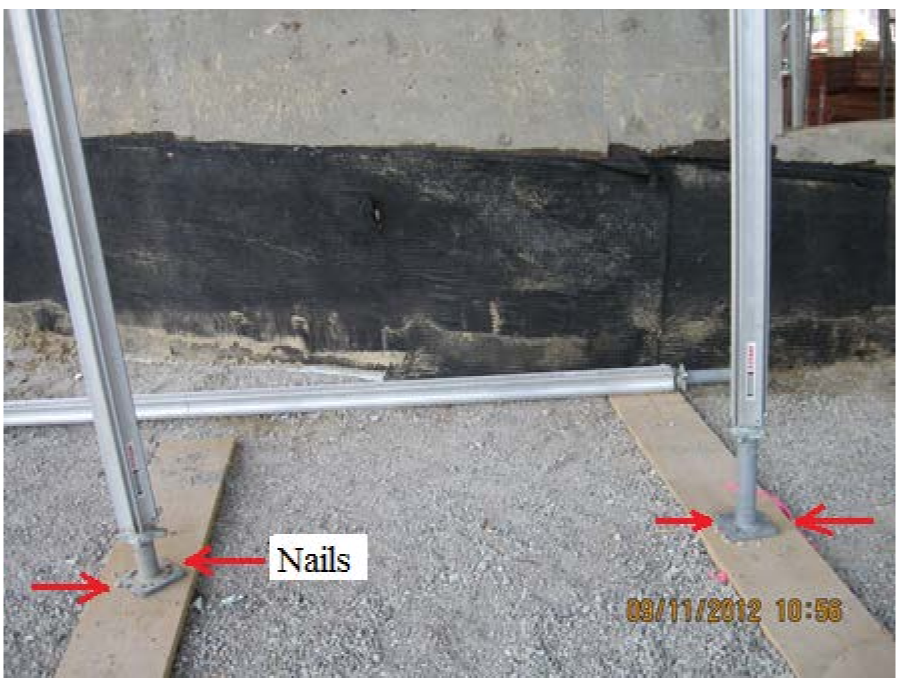

Figure 10. Base support condition of shores near the collapsed area.

Figure 11. Another view of the base support of post shores.

3. The contractor did not install the ledger frames (gates) at the perimeter of the building under the cantilever beams as required by the preliminary shoring drawings. Without ledger frames, the lateral stability of the shoring system could not be properly maintained.
4. The contractor did not install any lateral and diagonal braces at the splice level of post shores in the collapsed area, as required by OSHA regulations.
5. Thirteen post shores failed in the collapsed area. From the exposed post shores above the hardened concrete, the failure occurred at the splice joint near their mid height (Figures 12 through 15).
6. Those shores that only partially failed around the collapsed area were also bent at the mid height where the splice was located.

Statements from the Shoring Manufacturer

We discussed the shoring failure with the shoring manufacturer and the preliminary shoring designer, Gunter Sengel, Ischebeck USA, Inc., on September 13, 2012 and on January 18, 2013. The main points of discussion are summarized as follows:

Figure 12. Condition of the splice plates after the failure of the post shore.
Note that the two splice plates were located at the mid-height of the post shore.

Figure 13. Failure condition of the welded connection at the end of a post shore.

Figure 14. Another view of the failure condition of the welded connection at the end of a post shore.

Figure 15. Failure condition of the welded connection at the splice plate.
Note that this splice plate was located at the mid-height of the post shore.

1. Ischebeck stated that the nine sheets of preliminary shoring drawings, dated June 12, 2012 and July 3, 2012, were prepared by Ischebeck,. However, it was clearly marked in the Drawing Status Box of every sheet as "preliminary details only, not for construction, but issued for informational purposes only".
2. Ischebeck noticed when he visited the site after the incident that the shoring at the collapsed area was not erected in accord with his preliminary shoring design. The changes included:
   - The ramp at the first level was not installed. As a result, the length of the post shores was significantly increased from 13'-6" to 20'-1", and the shores changed from a single tier to two tiers.
   - The 11'-6" long cantilever main-beams perpendicular to the perimeter of the building under the working alley were replaced by two 5'-7" long simple main-beams. As a result, a post shore had to be relocated to the outer end to provide support to the second beam.
   - The ledger frames (gates) perpendicular to the perimeter of the building under the cantilever beams were not installed. According to Ischebeck, the ledger frames were the primary source of maintaining lateral stability of the shoring system against wind.
3. As per our request, Ischebeck also provided us with the following information;
   - Load test results with photographs of the HV Maxi legs for six different leg heights conducted at the Material Testing Laboratory of Lehigh University, PA.
   - Engineering properties of the HV Maxi legs (proprietary information).

   Based on the above information, we performed an analysis on the post shore (HV Maxi Leg). It is included in the next section.

5. Analysis and Discussion

Based on the above observations and descriptions, it is certain that the key structural members contributing to the collapse of the supporting system was the single post shores, one placed over the other, without any lateral bracing at the splice junction of the two tiers. The unbraced height of the post shores at the collapsed area averaged 20'-1". In our analysis, we considered the following properties. Actual numbers are not mentioned as they are proprietary.

- The aluminum alloy and temper of the extruded post shore.
- The yield tensile or compressive strength (F_y).
- The cross-sectional area (A).
- The moment of inertia (I_x).
- The section modulus (S_x).
- The modulus of elasticity (E).
- The radius of gyration (r_y).

Load Capacity (Elastic Buckling Load) of the HV Maxi Leg from its Stiffness

Based on the conditions of the post shores at the incident site, both the top and the bottom connections were considered hinged. Therefore, the effective slenderness factor (k) was assumed to be 1.0. The slenderness ratio (kL/r_y) was calculated to be 225, which was significantly greater than the slenderness limit of 66 (Reference 2, Page 99). Therefore, the 20'-1" high post shore was categorized as a slender column. The compressive strength of the slender column is governed by the Euler's formula. This strength, also called the elastic buckling stress (F_e), was calculated to be:

$$F_e = \pi^2 E / (kL/r_y)^2 = 1.97 \text{ ksi.}$$

The elastic buckling load (P_e) of the post shore was computed to be:

$$P_e = 7.19 \text{ kips.}$$

This elastic buckling load compared closely with the failure load of the load test results discussed in the next section.

Load Capacity the HV Maxi Leg from the Load Tests

In 2005, the shoring manufacturer, Ischebeck, conducted 16 load tests on the HV Maxi legs for six different leg heights at the Material Testing Laboratory of Lehigh University, PA. The test setup is presented in Figures 16 and 17; the base plate of the lower screw jack rested snugly on the bottom steel plate of the universal test machine. The base plate of the upper screw jack was also snugly fitted to the top steel plate of the test machine. According to Ischebeck, the top and

Figure 16. Load test setup of the single tier post shore at Lehigh Testing Laboratory (Modified from Ischebeck, 2005).

Figure 17. Load test setup of the two tiers post shores at Lehigh Testing Laboratory (Modified from Ischebeck, 2005).

bottom base plates were placed inside the taped area to ensure plumbness of the post shore. It should be noted that the bottom steel plate of the test machine was fixed against rotation and translation. The top steel plate was hung through a ball bearing to simulate the hinged condition. During the test, the top plate was moving up and down to apply the load. The purpose of this arrangement was to eliminate any flexural stresses in the specimen during compression or tension.

The standard loading procedure as per ANSI /SSFI SH300 (Reference 5) was used for the load tests of post shores. According to Ischebeck, the ultimate (failure) load of the post was the load when the post could not support any additional load. The photographs of the load tests indicated that the post shores had undergone flexural deformation at its mid height when subjected to the failure load. We noted, however, that the failure load of a post shore from the load test was uniformly greater than its elastic buckling load. Table 1 below explores the potential causes of this discrepancy.

Table 1

Failure Loads and Elastic Buckling Loads under Different End Conditions for HV Maxi Leg at Different Heights.

Column 1	**Column 2**	**Column 3**	**Column 4**
Height of the Post Shore (in.)	**Average Failure Load from the Load Test (kips)**	**Elastic Buckling Load (kips) in Hinge and Fixed Condition**	**Elastic Buckling Load (kips) in Hinge and Hinge Condition**
126 (10'-6")	44.7	40.9	26.2
144 (12'-0")	37.1	31.4	20.1
166 (13'-10")	24.8	23.8	15.2
174 (14'-6")	21.2	21.6	13.8
218 (18'-2")	13.5	13.7	8.76
241 (20'-1")	9.7	11.2	7.19

Notes: (1) All post shores are single tier, except for the 20'-1" high post shores with two tiers.
(2) The average failure load is the average of three tests, except for the 10'-6" high post shore which had only one test.
(3) Columns 3 and 4 are theoretical calculations of the elastic buckling loads with solid lengths of post shores without splice joints.

From the above table, we noted that the average failure load of a post shore (Column 2 of the above Table) matched close with the elastic buckling load in the top hinge and bottom fixed post shore (Column 3) of the same height. Particularly for the post shore heights of 166", 174" and 218"; the variations were within 5%. Due to the top and bottom support conditions, a slenderness factor of 0.80 was considered in Column 3. However, at the incident site, the top and bottom of the post shores were both hinged, increasing the slenderness factor to one. The elastic buckling load for the actual site condition was calculated in Column 4. Since neither load factors nor strength reduction factors were applied to the elastic buckling load of Column 4, it must be considered as the ultimate load. Thus, the failure load of a 20'-1" high unbraced post shore (HV Maxi Leg) could not exceed 7.19 kips. The effect of the splice joint in the two tiers post shore is discussed in the next section.

HV Maxi Leg Splice at Mid-Height

As observed at the collapsed area, all failed post shores were two tiers with their splice joints at mid-height. Ischebeck's load test program in 2005 included only three tests on two-tier post shores, the total height of the shores were about 20'-1". However, in the load tests, the splice joints were located either 2' above or 2' below the mid-height of the post shores. The test program did not include any post shore with a splice joint at its mid-height as was the case at the incident site. The load capacity of a column depends on its flexural capacity at the mid-height. Further, if the welded connection between the post shore and its splice plate was not of a full strength weld at the mid-height as was the case at the incident site, the load capacity of the post shore would be further reduced.

We considered the following observations and information:

- All the exposed post shores visible above the hardened concrete in the collapsed area failed at the splice joint near their mid-height. The partially failed post shores around the collapsed area were also bent at the splice joint near their mid-height. Thus, the splice joint at the mid-height was a weak link of the two tiers post shore.
- The welds were only applied around the exterior face of the post shore at the post shore and splice plate connection. There were no welds on the interior face of the post shore and the four grooves. Thus, this connection was not a full strength welded connection.

- The post shore manufacturer, Ischebeck, also confirmed that they did not provide a full strength welded connection between the post shore and its splice plate.
- From Table 1, for the post shore at a height of 241" with a splice connection about 2' from its mid-height, the load capacity was reduced by approximately 13% (1- 9.7/11.2) from its theoretical value.

We estimated that, if a splice joint was located at the mid-height, the load capacity of the post shore would be further reduced from approximately 13% to 15% to 6.11 kips (7.19 kips x 85%). Thus, the failure load of a 20'-1" high unbraced post shore (HV Maxi Leg) with a splice at mid-height is reduced to 6.11 kips.

Total Leg Load at the Time of the Collapse

Given the observed condition of the shoring system, the weight of the concrete form and support leg was calculated to be 270 pounds per leg. The weight of the wet concrete was computed to be 4,050 pounds (150 pcf x 0.75 ft x 36 ft^2) per leg. The construction live load was estimated to be 1,800 pounds (50 psf x 36 ft^2) per leg. Thus, the total leg load was estimated to be 6,120 pounds (6.12 kips).

Based on the previous information, the load-carrying capacity of a 20'-1" high post shore (HV Maxi Leg) with a splice at mid-height without any lateral bracing at the mid-height was estimated to be 6.11 kips. This capacity was very close to the estimated actual leg load of 6.12 kips. Thus, there was barely any margin of safety, and any installation inaccuracy, disturbance, vibrations, or placement of a large amount of concrete directly over a post could trigger the failure of the post shore. In addition, if one post shore failed, it would cause the collapse of the entire area of the forming system under the wet concrete. This was what we observed at the incident site (Figure 1).

HV Prop Loading Graphs

Ischebeck developed the HV Loading Graphs (Reference 6, Page 11) for the single post shore without lateral braces from the load tests at Lehigh Testing Laboratory, PA. Ischebeck then applied a safety factor of 2.5 to 3.0 on the failure load to obtain the allowable axial load. As described earlier, the setup of load tests had a fixed support at the bottom, which did not reflect the actual support condition of the post shore in the field. In addition, the load tests did not consider a weak splice joint at the mid-height of the post shore. Both of these factors would reduce the actual failure load from the load obtained from the laboratory tests. Therefore, Ischebeck's published HV Prop Loading Graphs overestimated the allowable axial load of the post shore.

Discussion

It is our understanding that Mr. Sengel of Ischebeck was the only shoring designer for the project before the September 6 incident. He prepared nine sheets of preliminary shoring drawings for information purposes only, and they were marked "not for construction". Apparently, the general contractor responsible for the concrete work did not prepare a set of shoring drawings that could be used for the construction.

Mr. Sengel stated that he noticed the changes in the erected shoring system from his preliminary design when he visited the collapsed area after the incident on September 7, 2012. He did not have an opportunity to inspect the erected shoring system before the concrete work started. Thus, the erected shoring system at the collapsed area was neither designed by a qualified designer nor inspected by an engineer qualified in structural design.

Our field observations indicated that all the post shores in the collapsed area failed at the mid-height splice location. There were no lateral or diagonal bracings installed at the splice level where the failures occurred.

As described in the previous sections of this report, the HV Prop Loading Graphs developed by Ischebeck had overestimated the load capacity of the post shores. This was mainly due to the fact that the bottom of the post shores tested in the laboratory was "fixed". This condition was not consistent with the actual condition in the field, i.e., a hinged support instead of fixed support at the bottom. The post shore with hinged connections at the top and bottom would have reduced load-carrying capacity. In addition, the load test did not include any post shore with a splice located at its mid-height. If welded connections at the splice joint were not at full strength, the load capacity would be further reduced.

Based on our calculations, the contractor applied an axial load of 6.1 kips per post during the placement of the wet concrete. This load exceeded the allowable load of 3.8 kips/post shore based on HV Prop Loading Graphs. Thus, the contractor exceeded the published allowable axial load by 60%. If the above factors are taken in consideration, the allowable load would have been even lower than the published values, and therefore the contractor exceeded the allowable load by even a larger margin.

The contractor supported single post shores on 2x12 planks (mudsill) on 4 in. thick crushed rock placed over compacted fill. The adjoining planks were not interconnected to each other to prevent potential lateral shifting. The contact pressure on the crushed rock and compacted fill was estimated to be around 6,000 pounds during the placement of the concrete. This value may have exceeded the allowable bearing capacity of the compacted fill. In addition, because of the noncohesive nature of the crushed rocks, the post shores were susceptible to differential settlement and tilting. The size of the mudsill was not correctly proportioned to reduce the contact pressure and to enhance the stability of the concrete forming system.

6. Conclusions

1. Shoring drawings or plans were not prepared for the project. The shoring plans of June 2012, prepared by Ischebeck were clearly designated as “not for construction”. They were issued for informational purposes only. This was a violation of OSHA Standard 1926.703(a)(2).
2. The contractor used single post shores, one on top of another, which were neither designed by a qualified designer nor inspected by an engineer qualified in structural design. This was a violation of OSHA Standard 1926.703(b)(8)(i).
3. The contractor did not brace the single post shores comprised of two tiers in two mutually perpendicular directions at the splice level. Also, the contractor did not brace the tiers diagonally in the same two directions. This was a violation of OSHA Standard 1926.703(b)(8)(iv).
4. HV Prop Loading Graphs developed by Ischebeck in their shoring catalog could be misleading in that they are not backed up by laboratory test data for a height of 20’-1"

with a splice joint at mid-height and with proper end conditions. Lehigh Laboratory load test data for the single post shores did not include any test data for the HV Maxi legs in two tiers with a splice at its mid-height.

5. The contractor claimed that Ischebeck's "Concrete Forming and Supporting Systems" catalog was used in this project. However, the contractor exceeded the allowable axial load published in the catalog by 60%. (Actual load of 6.1 kips versus allowable load of 3.8 kips).
6. The contractor supported single post shores on 2x12 planks (mudsill) on 4 in. crushed rock placed over compacted fill. The adjoining planks were not connected to each other. Because of the noncohesive nature of the crushed rocks, mudsill and eventually the posts were susceptible to displacement during the placement of the concrete.

7. References

1. Aluminum Design Manual, The Aluminum Association, 1994.
2. Aluminum Structures, J. Randolph Kissell and Robert L. Ferry, John Wiley & Sons, Inc., 1995.
3. Safe Practices for Erecting & Dismantling of Frame Shoring, recommended by Scaffolding, Shoring and Forming Institute, Inc.
4. Recommended Frame Shoring Erection Procedure, Scaffolding, Shoring and Forming Institute, Inc.
5. ANSI/SSFI SH300-2007 Standards for Testing and Rating Shoring Equipment, Scaffolding, Shoring and Forming Institute, Inc.
6. HV Slab Forming System, published by Ischebeck USA Inc., 2009.
7. Megashore Shoring System, published by Ischebeck USA Inc., 2009.