

# Investigation of the December 19, 2008 Collapse of Atlanta Botanical Garden Canopy Walkway during Construction in Atlanta, GA

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U.S. Department of Labor  
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# **REPORT**

## **Investigation of the December 19, 2008 Collapse of Atlanta Botanical Garden Canopy Walkway During Construction in Atlanta, GA**

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## **REPORT**

### **Introduction**

On December 19, 2008, a construction employee was killed and several other employees were injured when a canopy steel bridge under construction at the Atlanta Botanical Garden in Atlanta suddenly collapsed. At the time of the incident, concrete was being poured over the bridge deck. The collapse was massive, involving over seventy percent of the bridge. This was the first day that concrete was being poured. At the time of the collapse, the entire bridge structural frame was being supported over fifteen temporary shoring towers at approximately 30 ft. on centers.

### **Description of the Project**

Atlanta Botanical Garden (ABG) retained Jova/Daniels/Busby, Architects of Atlanta, GA to design the canopy walkway (Canopy Bridge) for their facility at 1345 Piedmont Avenue, Atlanta, GA. Halvorson and Partners, P.C., also of Atlanta, were the structural engineers. The architects and engineers designed the canopy bridge with inspiration from the famed Spanish architect, Santiago Calatrava, as an S-curved walkway rising from one end from the ground to a height of 35 feet and then sloping down to the other end. The canopy bridge was to be supported by cables strung from five masts which were off-set from the bridge. Near the center of the bridge was a fixed span, 70' long supported by inclined V-shaped columns. The canopy bridge was approximately 575' long. The width of the bridge varied from 11' to 18', providing overlookers to the visitors of the garden.

Hardin Construction Company, LLC (Hardin) of Atlanta, GA was the general contractor of the project. Hardin contracted with Steel Fab, Inc., of Norcross, GA to fabricate and erect the steel structure for the canopy bridge. Hardin also contracted with SDC Concrete Construction of Atlanta, GA to place rebars, mesh, and to pour the 6" concrete for the bridge deck. Steel Fab, in turn, retained Williams Erection Company of Smyrna, GA to erect the structure. Steel Fab prepared the drawings and fabricated the structural steel and delivered it to the site for Williams to erect. Williams was required to temporarily shore the canopy bridge during construction until

all the construction had been completed, at which time the canopy bridge would be supported by cables and masts. To erect the temporary shoring towers, Williams contracted with Southeast Access, LLC, of Acworth, GA. Williams also retained LBYD, Inc., structural engineers, to design the shoring towers. Williams also retained Atlas Piers of Atlanta, Inc. (Atlas) to install helical anchors in the ground to support the shoring towers. Helical anchors were recommended by LBYD, see Fig. 1.

Structural drawings required that the temporary shoring towers be placed at a spacing not exceeding 30'. Atlas began to install helical anchors at locations indicated by Williams. Eight anchors were provided for each shoring tower. Some of the anchors had to be re-located when they hit tree roots. By October 28, 2008, all of the anchors were in place. Southeast leased the scaffold frames, bracings and steel beams from a company, Shore All of Georgia, and began erecting the shoring towers. Williams started to erect the structural steel framing of the canopy bridge on September 15, 2008 and completed the erection of the last segment on November 25, 2008. Each segment was approximately 30' long. Each segment had all of the secondary framing members pre-assembled in the shop before shipping to the site. The secondary members consisted of framing for the overlookers, splice plates, etc. No assembly was performed at the site except for bolting the main bridge pipe at the splice locations. The sequence of erection of the structural segments was segments 2 thru 12, 14 thru 16, 21 thru 18, 13 and finally 17, see Figs. 8 and 16. The erection proceeded without any known problems. Thereafter, Williams began to lay the metal deck over the overlooker framings, see Figs. 2 thru 7.

Having completed the erection of structural steel and placement of the metal deck, SDC started placing concrete by pumping from the concrete trucks onto the bridge deck, beginning a few feet south of shoring tower #5 and proceeding towards tower #8. There were at least twenty employees engaged in pouring the concrete and finishing the deck. They successfully placed concrete between towers #5 and #7. They had just reached tower #8, when the structural framing of the canopy bridge suddenly collapsed. All of the employees located over the deck fell with the collapsing framing, resulting in one fatality and several injuries.

The collapse of the canopy bridge to the ground covered an area beginning at the fractured splice near shoring tower #5, and extending all the way to the northwest abutment. The canopy structure south of tower #5 remained intact up to the northeast abutment. Three splices, near towers #5, #7 and #9 sheared off at the bolted connections. Another splice near tower #13 was partially fractured. During the collapse, the main structural pipe was laterally displaced approximately 12" from its original alignment towards the south. The main pipe did not exhibit any torsional deformation. Shoring towers #6 thru #15 collapsed. Shoring tower #5 remained partially standing. The rest of the shoring towers also remained standing.

### **Structural design of the canopy bridge**

The bridge featured a primary member consisting of a 30" diameter,  $\frac{3}{4}$ " thick, HSS structural pipe (main pipe) laid out in the desired s-curved shape to be supported by cables at intermediate locations. The bridge was supported at each end by concrete abutments. Near the center of the bridge there was a fixed straight span 70 feet long and 11 feet wide, supported by two sloping pipe columns, 16" in diameter and  $\frac{5}{8}$ " thick, forming a V, see Figs. 5 and 7. In addition, there was a stub column placed near the north abutment. The bridge gradually rose from the north abutment to its highest point near the center and then gently sloped down to the south abutment. Various overlookers extending to either one side or both sides of the pipe were installed on the bridge. The framing of the overlookers generally consisted of cantilevered members, 8"x 8" tubes, approximately 10' apart, welded to the pipe at 2'-8" above the center of the pipe. The tubes were diagonally braced to the pipe by 6" diameter pipes. A 2" deep steel deck spanned over the tubes. 6" lightweight concrete was to be poured for the deck. The top of the concrete deck was approximately 3'-6" above the center of the pipe. For a typical overlooker framing, see Figs. 4 and 5.

Four cable masts sloping away from the pipe, marked B, E, F and G were placed inside the arc of the bridge. The masts consisted of 24" diameter,  $\frac{1}{2}$ " thick HSS structural pipe. The height of the masts varied. Generally, the top of the mast was 25 feet away from the pipe, and the bottom of the mast was 9 feet away. From the top of each mast, three stay cables (marked 1, 2 and 3) sloped to the gusset plates of the main pipe member to support the bridge, see Fig. 3. The gusset

plates connecting all three stay cables were generally placed at the location of the splice connection. Cables 1 and 3 were connected in an angular direction to the pipe. Cable 2 was connected to the pipe in a perpendicular direction. From the top of the mast, two other cables (backstay cables 4 and 5) sloped in the opposite direction and were anchored to the ground; see Fig. 3 for a typical layout.

The gusset plate connecting cable #2 and the pipe also had a vertical tie down cable, identified as cable #5, some 9 feet away from the main pipe. The bottom of cable #5 was connected to the mast near its base. The gusset plate of cable #2 also had a damper rod assembly attached to it. The damper rod was connected to the base of the mast.

### **Evaluation of the structural design of the canopy bridge**

Analyses were performed to determine whether the structural design of the bridge met the industry standards if it was constructed according to the contract drawings. A decision was made to identify inherent design flaws, if any, in the structure which could have contributed to the collapse.

STAAD.Pro v8i program was used in the analysis. The geometry of the bridge was taken from the structural drawings S-0 thru S-8 containing a number of revisions, the latest being on July 24, 2008. The bridge was modeled with the deck at varying elevations, as per the drawings. The sizes of all structural members with their yield strengths were derived from the drawings. Masts B, D, E and F were modeled at locations indicated on the drawings. Three cables from each mast supporting the main structural member were provided with an approximate initial tension. Also, an additional hold down cable was placed at the main structural member at the center cable of each mast. The elevations of the mast and the slopes of all supporting cables were taken from the structural drawings. The bridge was also modeled with a short vertical leg near the northwest and northeast abutments and two sloping V-shaped legs near the middle of the bridge.

Shop drawings prepared by Steel Fab indicated that the weights of the gusset plates, splice plates and rings, the continuous  $\frac{3}{4}$ " plate welded to the primary member, and other items amounted to

approximately 50% of the dead load of the main structural member (30" pipe). Therefore, an additional 50% weight was added to the primary structural member. For the out looker framings, the shop drawings indicated the weights of all attachments to be approximately 20%. So, 20% weight was added to the overlooker framings. Overlookers with varying dimensions were modeled. The dead load of concrete over the deck was also taken into account.

The analysis was based on a live load of 85 psf uniformly placed over the entire bridge. A reduced live load of 65 psf is also permitted. In the analysis, alternate placement of the live load was not considered. Wind or seismic loads were also not considered. As mentioned earlier, an approximate initial tension was provided for the cables. It was determined that the bridge design generally met the industry standard for the code prescribed loads. None of the structural members were overstressed, and they met the design criteria.

However, an area of concern was discovered in regard to the deflection of the cantilevered overlookers. The industry standard is to limit the cantilever live load deflection to  $L/300$ . A review of the buckling computations prepared by the SER indicated that the cantilevered overlookers deflected greater than the allowable value under a reduced live load of 30 psf and under reduced cable tension. It is believed that the relative deflection between the bridge's main pipe and the corresponding tip of the overlooker would have been similar under the full live load of 65 psf and the final tension of the cables. It is estimated from the SER's computations that the cantilevered overlooker would deflect 1.27" under the full live load, well in excess of the allowable deflection. The noncompliance of the deflection criteria with the industry standard, however, did not contribute to the collapse, but would have become a serious serviceability issue upon the completion of the bridge.

### **Design of Shoring System**

Williams Erection Company (Williams) retained LBYD, Inc. of Atlanta, GA, to design temporary "shoring towers" to support the bridge's structural members during erection of the bridge. These temporary shoring towers were to support the bridge until concrete was placed on



the deck, and all permanent masts with cables to support the bridge were erected. After the bridge was self-supporting, these shoring towers were to be removed.

There was no formal written contract between Williams and LBYD. They have both worked together on a number of other projects in the past. LBYD contends that they were not retained to actually “design” the shoring towers, but to select certain shoring tower posts and related members from shoring catalogs. However, LBYD agreed that they did design the supporting structural beams at the top and bottom of the shoring towers. LBYD did not prepare formal drawings of the shoring towers, but provided a number of sketches to Williams to convey the intent of their design. Besides, there were a number of transmittals and letters between them indicating LBYD’s requirements for the shoring towers, including soil anchors.

Williams provided a layout of the bridge with approximate locations of the shoring towers. LBYD’s original design is contained in their letter of November 26, 2008 with an attached graphic of the shoring towers and the arrangement of the top beams. This letter was revised on December 2, 2008, see Figs. 8 thru 10. The revision was prompted by the fact that Williams preferred to use 4’x5’ scaffold towers instead of 6’x6’ towers as originally proposed by LBYD. The 5’ dimension was parallel to the bridge’s main pipe, and the 4’ dimension was perpendicular to the pipe. The clear spacing between the towers was proposed to be 6’. The center-to-center dimension between the scaffolding towers were therefore 10’. The shoring towers consisted of two scaffolding towers, each supported at the base of the towers by four W8x10 beams placed over the soil anchors. LBYD required that “the beams are connected to each other and to the posts and anchors with Waco beam anchors.” It further required that “adjacent anchors are braced to each other with 3x3x1/4” angles, where the sloping terrain results in unbraced anchor shafts above grade.” The beams at the top of the shoring towers consisted of W10x33, later changed to 2-W10x19 beams, placed directly underneath the main pipe bridge member. The 2-W10x19 beams were to be supported by W10x19 beams at each end, spanning 5’ parallel to the bridge’s main pipe. These beams were, in turn, supported by four W8x10 beams, see Fig. 10. LBYD permitted the center of the bridge pipe to be within 10% of the mid-span of the W10x33 or 2-W10x19 beam.

For the design of the W10x33 and W10x19 beams, LBYD computed a maximum load of 28 tons, with the average being 22 tons. This included the lightweight concrete bridge deck, and a construction live load of 25 psf. LBYD computed the maximum anchor reaction to be 10 kips, less than the assumed anchor capacity of 11.25 kips, based upon the logs provided by Atlas. However, it must be noted here that the anchor capacity of 11,250 pounds is the ultimate capacity, and not the allowable load, see Atlas' letter of November 5, 2008. LBYD asked for an allowable capacity of 10,000 pounds in their letter of November 21, 2008. The Atlas log sheet indicates that all the soil anchors met LBYD's requirements.

### **LBYD Design of Shoring Beams**

Based upon the original layout of the shoring towers provided by Williams, LBYD estimated the maximum load on the shoring tower to be 28 tons, with the average being 22 tons. The beams supporting the bridge's main pipe were proportioned to be W10x33. After the incident, LBYD contended that their design was based upon the use of high strength steel conforming to ASTM A-992, though neither in their letter of November 26, 2008 (revised December 2, 2008) nor in the sketches provided to Williams did LBYD indicate the strength of the steel. LBYD argued during an OSHA interview that since the structural design of the bridge by the structural engineer of record was based upon the use of high strength steel, LBYD did not find it necessary to indicate the strength of the steel for the shoring tower steel beams. This argument is considered nonpersuasive because the design of the temporary shoring towers was independent of the design of the bridge. Besides, the design of the bridge and the temporary shoring towers were performed by two independent engineers, and the drawings were independent of each other. In the absence of a steel strength requirement by LBYD in the sketches and letters, Southeast ordered steel beams without mentioning either normal strength (A-36) or high strength steel (A-992). Some of the beams used in the temporary shoring towers were, therefore, A-36 steel instead of the higher strength as intended by LBYD.

OSHA examined the adequacy of the design of the beams based upon the loads indicated by LBYD in their letter of November 26, as revised on December 2, 2008. It was determined that the design was inadequate even when the use of high strength steel was considered. W10x33

could support combined dead and live loads of only 42.5 kips based upon LRFD requirements, as compared to the required 56 kips. In this analysis, an effective load factor of 1.36 (to account for variable load factors for dead and live loads), and a resistance factor of 0.9, as per AISC (American Institute of Steel Construction) and ASCE (American Society of Civil Engineers) were considered. If A-36 steel use was assumed, the maximum load that W 10x33 could support was determined to be only 30.5 kips. At the request of Williams and Southeast, LBYD approved the use of 2-W10x19 instead of one W10x33. This request was made because Shore All could not furnish W10x33 steel beams. OSHA also evaluated the load carrying capacity of beams without considering the load factors and resistance factors in which case the W10x33 and 2-W10x19 could support an ultimate load of 64.7 kips and 72.0 kips, respectively, see Fig. 36, if use of high strength steel was assumed. These ultimate loads are higher than the required strength of 56 kips but will not have any factor of safety, a gross deviation from the standard design practice. If A-36 steel use was assumed, W10x33 and 2-W10x19 could support an ultimate load of 46.5 and 51.8 kips, respectively, still below the required strength of 56 kips.

After the incident, LBYD performed a more detailed and refined load evaluation of the shoring towers. A copy of the computations was provided to OSHA. LBYD computed a maximum load of 56.4 kips on sequence 8. Further, LBYD reduced the construction live load as per ASCE provisions, based upon the area consideration, resulting in a total reduced load of 51 kips. Even with the reduced load of 51 kips, the design of W10x33 remained deficient as it could support a load of only 42.6 kips, see Fig. 36. The 2-W10x19 beams were also deficient.

The omission of the high strength steel in the drawings by LBYD proved to be inconsequential due to the findings of the testing lab, discussed below. The proportioning of all other beams was considered satisfactory.

### **Laboratory Testing of Shoring Beams**

Southeast Access which erected the shoring towers based on LBYD sketches, was not aware of the high strength steel requirement. During the OSHA interview, Shore All, which leased the steel beams to Southeast Access, indicated that they stored only the normal strength steel, and

not the high strength steel. OSHA retained TEC Services, Inc., Lawrenceville, GA (Lab) to perform tests to determine chemical and physical properties of six selected shoring beams (6SX02, 6SX03, 6SX09, 7SX08, 7SX09 and 8SX03) to determine yield and ultimate strength of the steel for compliance with ASTM A36 and ASTM A992. Approximately three-foot long sections were cut from each of the six beams and brought in the lab facility. Lab took measurements of the above beams. The lab noted in their report (see Attachment A) that the flange thicknesses were approximately 10% less than AISC published values. Two of the beams (6SX03 and 6SX09) were determined to be W10x12 instead of W10x19. The lab further determined that four beams, 6SX02, 6SX09, 7SX08, and 7SX09, conformed to high strength steel A-992, and the other two beams 6SX03 and 8SX03 conformed to A-36 steel. Although the beams 6SX03 and 8SX03 did not meet the requirements for ASTM A992, the yield strength of the beams were approximately 46ksi, significantly higher than 36 ksi. After the collapse, W10x12 (6SX03) was observed under the pipe.

### **Soil Anchors**

On August 28, 2008, LBYD recommended that Williams use segmented helical anchors, also known as helical piles, to support the shoring towers at the ground. The Botanical Garden wanted to minimize disturbance to the topsoil and organic layers at the ground level. Helical anchors were recommended in deference to the owner's requirements to achieve limited soil disturbance. Another advantage was that the anchor could be easily retracted if any tree root was encountered during its placement, and could be relocated.

Williams retained Atlas Piers of Atlanta, Inc. (Atlas) to install the piles. Atlas used an empirical method to determine the ultimate bearing capacity of the anchors. The method was based upon the assumption that "the torsional energy required to install a helical anchor/pile can be related to the ultimate load capacity of an anchor/pile". Atlas used chance brand SS-5 Helical piles, C110-0691, which they installed to support the shoring towers and other structural elements, see Fig. 15. The formula used to determine the axial allowable capacity of the anchors was as follows:

Allowable axial bearing capacity = 10 (Torque factor) x Installation torque / 2 (safety factor).  
 For example, a torque of 2,250 ft. lbs. would provide an allowable load of 11, 250 pounds.

The anchors used in the project had square 1.5"x1.5" solid shafts made of high strength steel (70 ksi) with three 8" plates in the lead sections. They were generally 7' long, with additional sections of 5' length, without plates, which were added if necessary. The installation lasted from September 8, 2008 through October 28, 2008.

Initially, Williams proposed to use 18 shoring towers, later reduced to 15. As mentioned earlier, each tower consisted of a pair of scaffolding towers with four legs for each tower. For each leg, an anchor was proposed with a total of eight anchors for each shoring tower. Atlas installed all the anchors, and had to reposition some of them when they hit tree roots. At the top of the anchors were steel plates to support the bottom steel beams for the scaffolding towers. There is no survey available to determine the elevations of the anchors before the incident. However, a survey was conducted after the incident, locating the anchors, elevations of the grade and the top of the anchors, see Fig. 29.

Fig. 31 indicates the anchors for towers No. 5 thru 10 with the depths of the embedment and the exposed height of the anchors above the grade. All anchors appeared to have adequate embedment. However, two anchors under Tower #7, and five under tower #8 had variable embedment depths ranging from 17" to 43". The following illustrates the depth of embedment for anchors under towers #7 and #8:

Tower # 8

8AHP10	8AHP11	8AHP12	8AHP13
<b>22"</b>	<b>43"</b>	<b>22"</b>	76"
8BHP10	8BHP11	8BHP12	8BHP13
<b>17"</b>	77"	<b>31"</b>	65"

Tower # 7

7AHP10	7AHP11	7AHP12	7AHP13
80"	80"	<b>35"</b>	<b>29"</b>

7BHP10	7BHP11	7BHP12	7BHP13
77"	74"	68"	63"

The above data is based upon the installation photos, post-incident measurements and a survey performed by Hardin. During an interview with OSHA on December 24, 2008, Atlas stated that approximately ten helical anchors had an embedment depth of only 3' and were projecting 4' above the grade. The helical anchors with blades sticking above ground for tower #7 and tower #8 can be seen in Figs. 32 and 33. Assuming that the top one foot or more consisted of topsoil and organic matter, the 8BHP10 anchor had an embedment depth of only 5". Anchors 8AHP10 and 8AHP12 had an effective embedment depth of only 10". Similarly, anchors 8BHP12, 7AHP12, and 7AHP13 had effective embedment depths of 19", 23", and 17", respectively.

While the axial gravity capacity of the anchors might not have been impaired due to shallow embedment, the lateral load capacity was certainly questionable. We have determined that the maximum lateral load capacity of such anchors could vary from zero to a maximum of 500 pounds. Anchors having effective embedment depths of 5" to 23" can not be relied upon to support the load, particularly because there was no sub-surface investigation performed at the site. Unequal depths of anchors supporting a scaffolding tower could result in differential settlements of the anchors.

For Williams to have cut the anchors without consulting with the geotechnical and structural engineers is considered to be a serious lapse of judgment, directly contributing to the collapse.

LBYD was informed that some anchors had insufficient embedment depths of approximately 3'. LBYD then prepared sketches for Williams to provide diagonal bracings between those anchors, and between the adjoining shoring towers in case even one anchor had insufficient embedment depth. LBYD was not informed of the anchor depths as low as 17". Nevertheless, Williams did not provide the diagonal cable bracings between the anchors to resist the lateral loads as required by LBYD, see Figs. 11 and 12. Williams however installed L3x3x1/4" horizontal bracings between anchors at tower #8A. Bracing were not provided at tower #7A. Therefore, these anchors were highly susceptible to lateral movement, jeopardizing the stability of the shoring

towers. Our analysis indicated that the lateral load to be resisted by the anchors in tower #7 and #8 was on the order of 10 kips.

## **Failure**

The shoring tower at grid D.1 (tower #5) was relocated to grid D, see Fig. 16, by Williams and this increased the spacing between tower #5 and tower #6 to 45', see Fig. 18. The original distance between tower #6 and tower #7 was 27'. The distance between tower #5 and tower #7 thus became 72'. This resulted in an average effective span of 36' for tower #6. Assuming that concrete was placed only between tower #5 and tower #8 and the construction live load could be reduced to 19 psf, the loads on the shoring towers # 6, 7, 8 and 9 were approximately 56, 42, 34 and 16 kips, respectively, at the time of the incident.

We analyzed which tower's failure could have triggered the bridge collapse. If shoring tower #8 is assumed to have failed first due to any reason, then the load from the tower #8 would have been transferred to the adjoining towers, i.e., tower #9 and #7 with tower #7 carrying a greater load of 65 kips. Tower #7 and tower #9 would have been able to support the increased loads as the loads were still below their ultimate capacity. Furthermore the bridge main pipe would have satisfactorily spanned between towers #7 and #9 without any overstress. Therefore, failure of tower #8 could not have triggered the bridge collapse.

Failure was therefore initiated either by tower #6 or #7. Failure of either tower would certainly have resulted in the failure of the bridge. If either tower failed, the resulting loads on the other adjoining towers would have been much greater than their ultimate capacities, and would have resulted in their failures. Consequently, the bridge's pipe would have had to support the loads by itself without tower #6, #7 and #8. This would have increased the loads on the pipe and would have resulted in failure of the pipe's splices near tower #5 and #9. Our computations indicated that the splices would have been subjected to direct shear, shear due to torsion and flexural stresses. The 28 bolts which were in double shear at the splice near tower #9 would have been subjected to a force much greater than their ultimate capacities. Similarly, the bolts in single

shear near tower #5 would also have failed under the increased loading. Therefore, failure of either tower #6 or #7 could have triggered the collapse.

We will first examine tower #6. As a result of the collapse, two steel beams, W10x12 and W10x19, were observed trapped under the bridge's main pipe, at tower #6. If Williams placed these two beams under the pipe, in the worst-case scenario, the two beams must have then jointly supported a load of 56 kips. Assuming that the load would have been shared by the two beams in the ratio of their stiffness, W10x12 and W10x19 would have been subjected to loads of 19.6 and 36.4 kips, respectively. Given the higher yield stresses of the beams as determined by the laboratory testing, the beams would have just been able to support the loads. If Williams did not place these two beams under the pipe, but rather placed 2-W10x19 under the pipe, the 2-W10x19 beams would have had enough strength to support the loads. W10x12s, if placed anywhere else would also have had enough strength to support the load. Post-collapse observations indicated that the W10x12 and W10x19 beams were placed by Williams under the pipe. The other W10x12 beam is believed to have been placed either over the scaffolding tower or across the scaffolding towers, was ejected during the collapse, and was found lying some 30' away from tower #6. Further, all eight soil anchors under 6A and 6B had adequate embedment depths. If it is assumed that tower #6 had, in fact, failed, triggering the collapse, then the load would have been instantly transferred to towers #7 and #5. Tower #7 would then have been subjected to an increased load of 98 kips, much greater than the combined strength of 2-W10x19 beams and subjecting them to failure in a flexural mode. Interestingly, post-collapse observations did not indicate such deformations in the beams. In fact, the beams remained practically undeformed. The failure of tower #6 can, therefore, be eliminated as a triggering cause.

We next examined tower #7. During post-collapse inspection, scaffolding tower #7A was observed to be severely deformed and damaged, while scaffolding tower #7B remained relatively intact and leaned over to one side. Besides, a substantial quantity of fresh concrete was observed to have fallen over #7A. Scaffolding tower #7A was observed to have sustained the most damage among scaffolding towers, #6A, 6B, 7A and 7B. Under scaffolding tower #7A, two of the four soil anchors on the west side had inadequate embedment lengths of 29" and 35" in the soil. If the top 12" of the soil is disregarded, the effective embedment depths of the anchors on



the west side were only 17” and 23”. The anchors on the east side of the scaffolding tower #7A were anchored 80” deep. The distance between the east and the west anchors was approximately 8’. The difference in the relative elevations of the bottom of the anchors between the east and the west were 51” and 45”. A review of the bottom elevations of the soil anchors of the towers indicated that it was highly unlikely that the west soil anchors of tower #7A with shallow embedment had hit a bedrock formation and that the anchors were resting on a sound rock stratum. Based on the micropile drilling records of the area by Hayward Baker Inc, dated June 20 through 26, 2008, a silty sand layer was first encountered with an average thickness of 24’. The average depth of bedrock was 45’. Thus, the anchors had hit some boulders below which could have been soft or loose soil subject to gradual settlement. If, in fact, there was a differential settlement between the east and the west anchors of scaffolding tower # 7A, the tower would have been subjected to eccentric loading, resulting in its collapse. Since the towers were not supported at a uniform reinforced foundation with slabs or similar systems which are effective in resisting differential settlements, the tower’s structural integrity could have been seriously jeopardized in the event of settlement of an individual anchor. It is suspected that the differential settlement of the soil anchors resulted in overloading of scaffolding tower #7A, thus causing the failure. The observation of the post-collapse failure supports this scenario, as explained above. With the failure of scaffolding tower #7A, tower # 7B failed, and subsequently, towers #6 and #8 failed due to increased loading. With the elimination of towers #6, #7 and #8, the bridge’s pipe was overstressed at the splices near tower #9 and #5, leading to the resulting total failure.

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## **Conclusions**

We reached the following conclusions:

1. A load higher than the design load was imposed upon shoring tower #6 due to increased spacing between shoring towers #5 and #6. The distance between shoring towers #5 and #6 was increased without any consultation with or drawings from the engineer who designed the shoring towers. No one performed any structural evaluation whatsoever.

2. The shoring contractor did not comply with the shoring tower design provided by the shoring tower engineer in that (a) three of the steel beams used in shoring towers #6 and #9 were discovered, after the incident, to be W10x12 instead of W10x19, as designed; (b) shoring towers with at least one helical anchor having inadequate embedment in the soil were not braced as required by the shoring tower design; (c) shoring towers 20' or greater in height, with at least one helical anchor with inadequate soil embedment were not braced properly as per the shoring tower design. This contributed to the collapse.
3. The shoring contractor did not provide the required diagonal cable bracings between the anchors which had insufficient embedment depths, as required by the shoring tower engineer. Anchors with shallow depths are highly susceptible to differential settlement and lateral movements. This created a serious hazard. The shoring contractor had received instructions to provide lateral bracings. Seven anchors under tower #7 and #8 had embedment depths that varied from 43" to 17". This omission contributed to the collapse.
4. The shoring tower contractor did not exercise the required standard of care in ensuring that all the steel beams that he received from the leasing contractor were, in fact, the same size beams he had ordered.
5. The shoring tower consultant designed the steel beams supporting the main bridge pipe member to be of high strength steel, ASTM A-992 but failed to indicate this requirement either in his sketch or in his communications with the shoring contractor.
6. The structural design of the beams of the shoring towers was flawed in that the beams supporting the bridge's main member were under-designed according to the recognized industry standards, even when the use of high strength steel is considered.
7. The structural drawings prepared by the structural engineer of record specifically indicated that the spacing between the shoring towers should not exceed 30'. The shoring contractor did not follow this requirement.

8. The structural design of the canopy bridge by the structural engineer of record was generally in compliance with the industry standards. The design, however, did not comply with the deflection requirements, creating potential serviceability concerns.

## ATLANTA BOTANICAL GARDEN CANOPY WALKWAY COLLAPSE

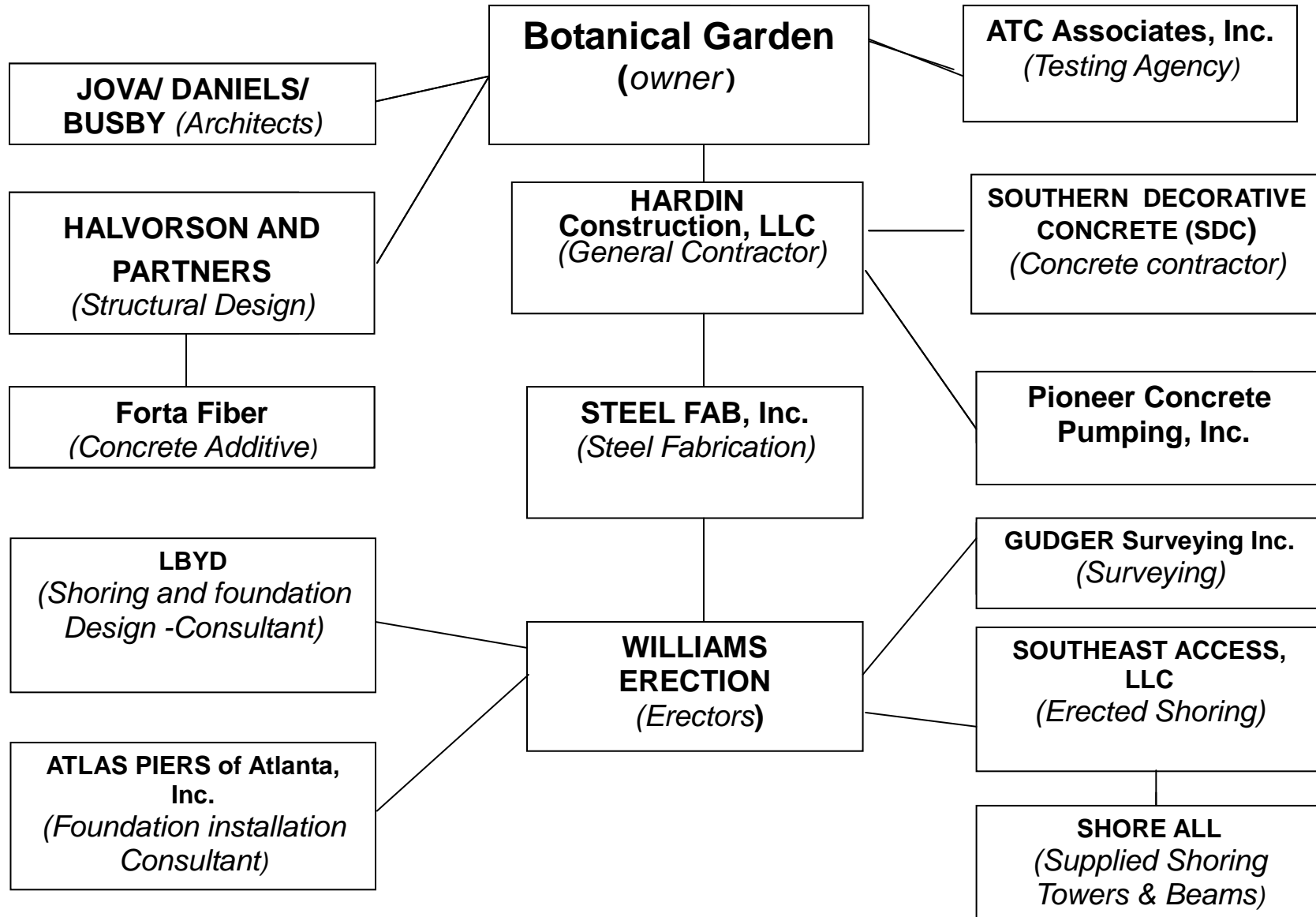


Figure 1 - Project Organization Chart

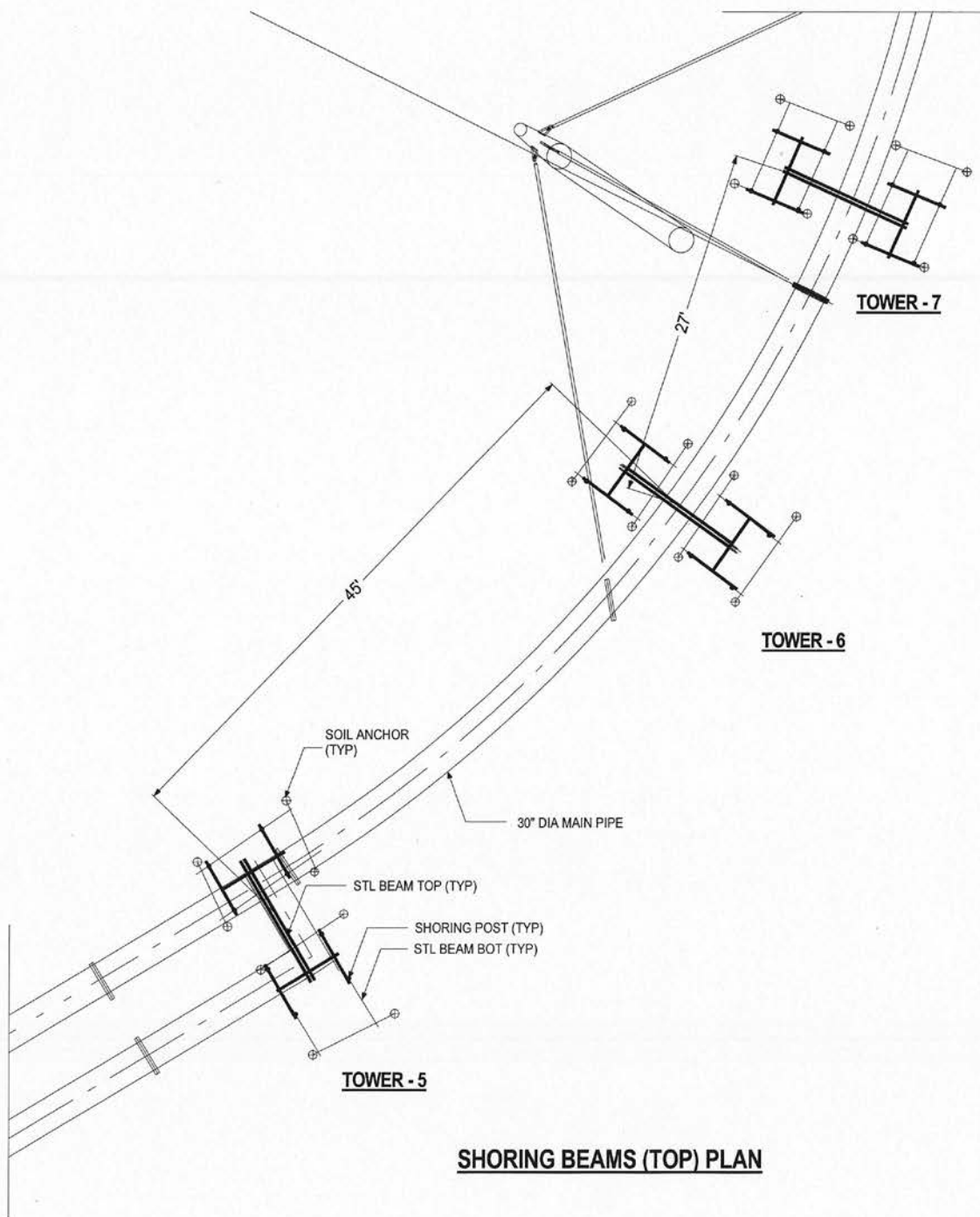
**Figures 2 to 15 (pages 21 to 34) are Drawings (not shown)**





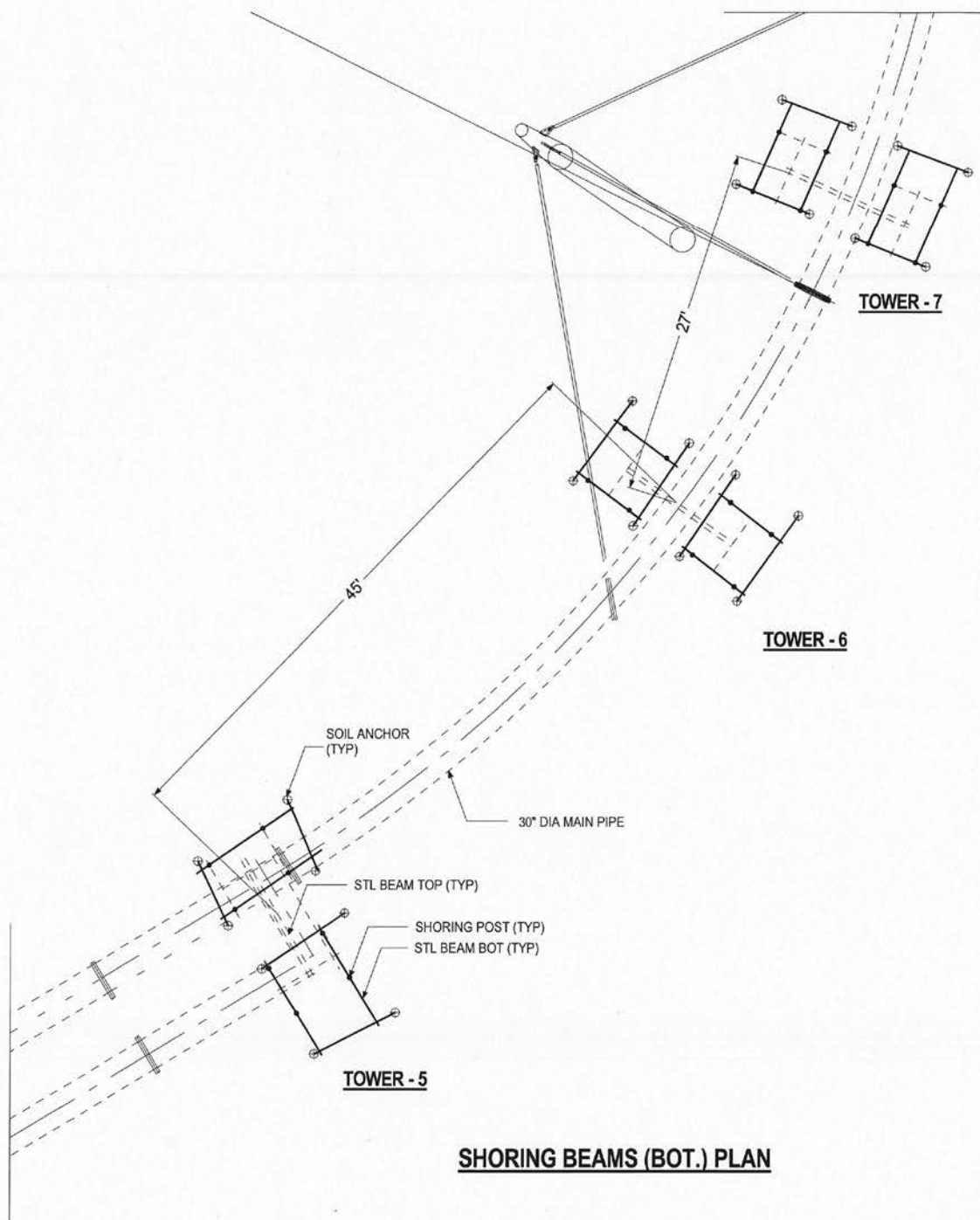


**Figure 17**

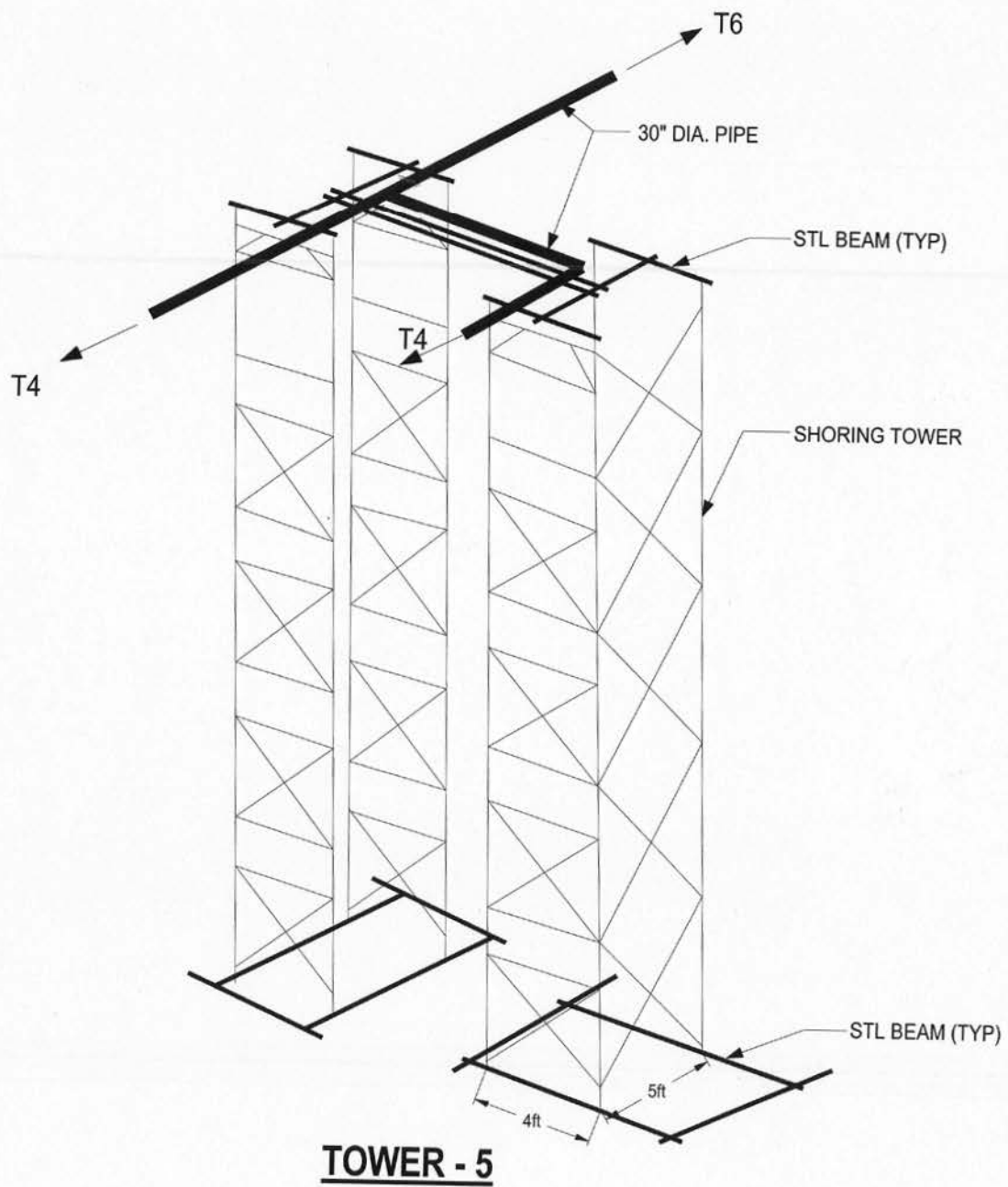


**Figure 18**

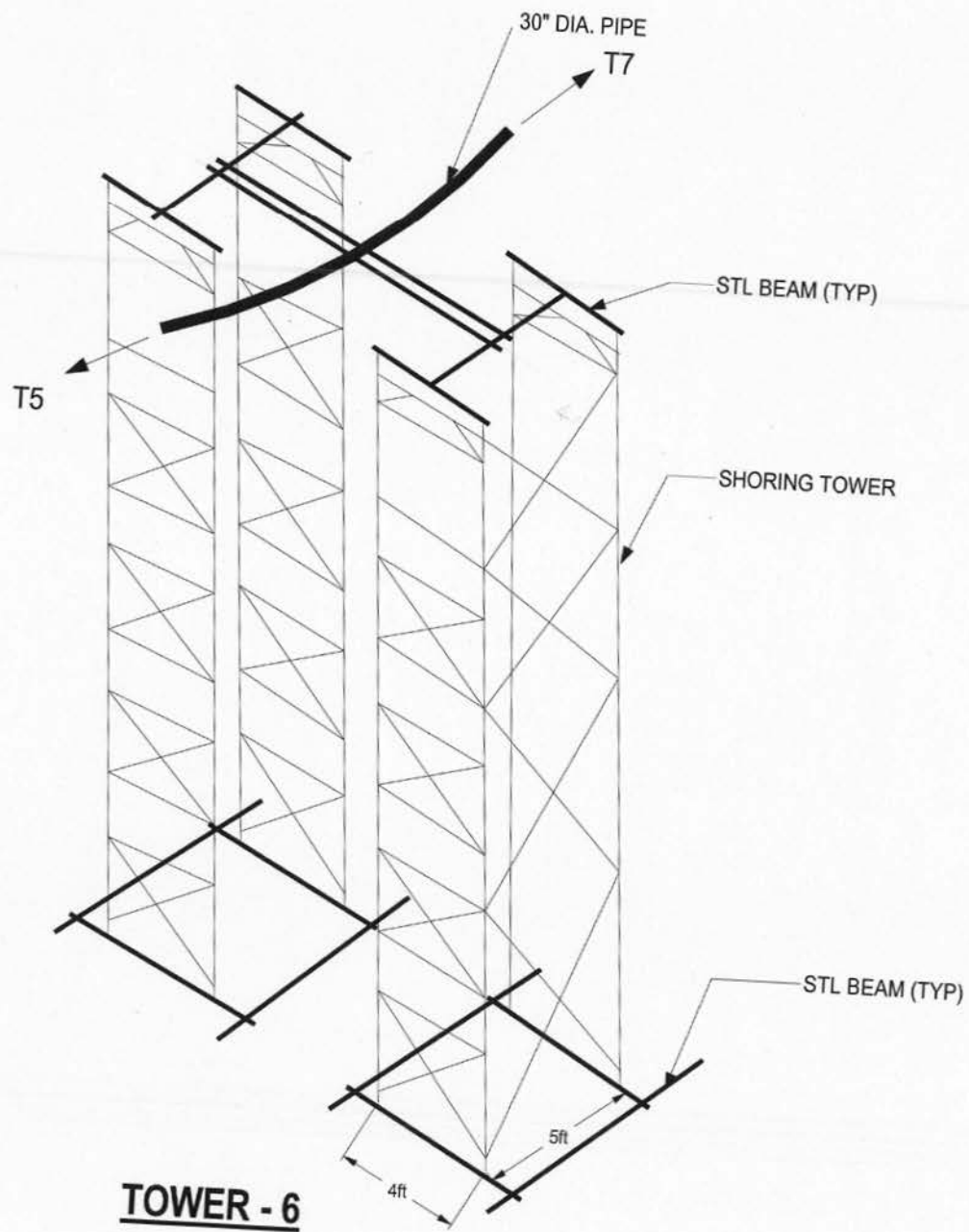




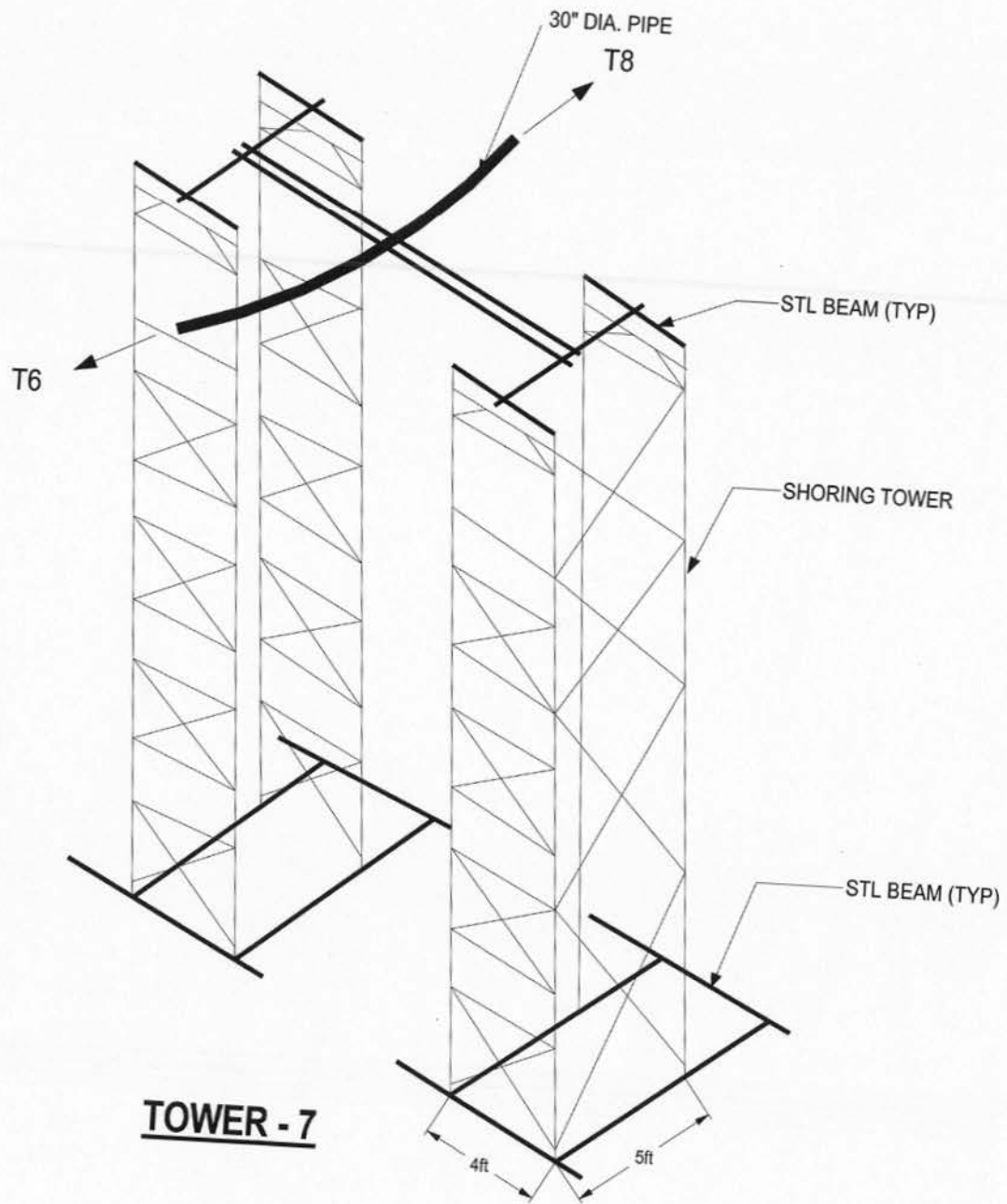
**Figure 19**



**Figure 20**



**Figure 21**



**Figure 22**





**Figure 23 Tower #6A Shoring beams after collapse**



**Figure 24 Tower #6B after collapse, No deformation**





**Figure 25 Tower #7A after collapse**



**Figure 26 Tower #7A after collapse**

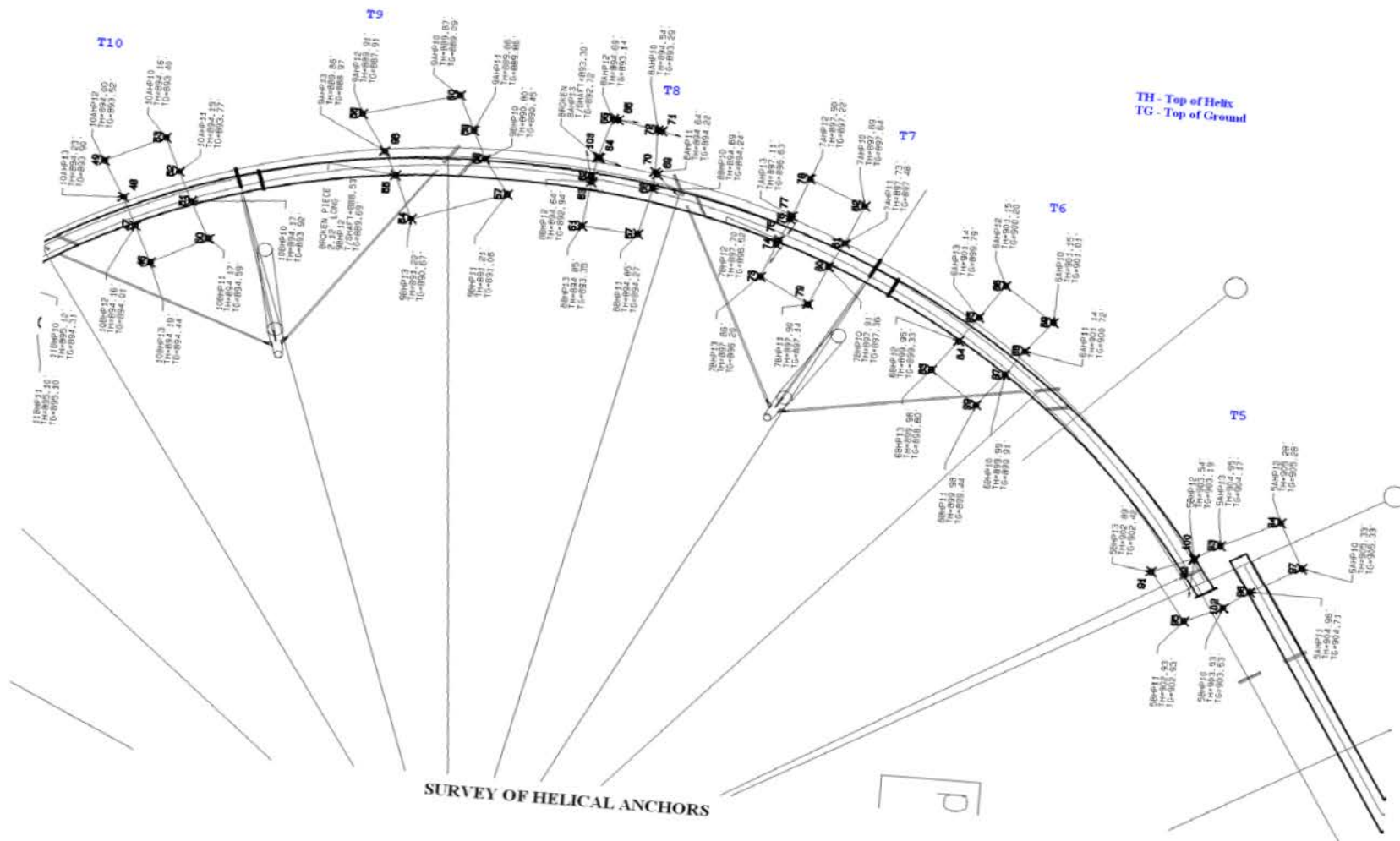




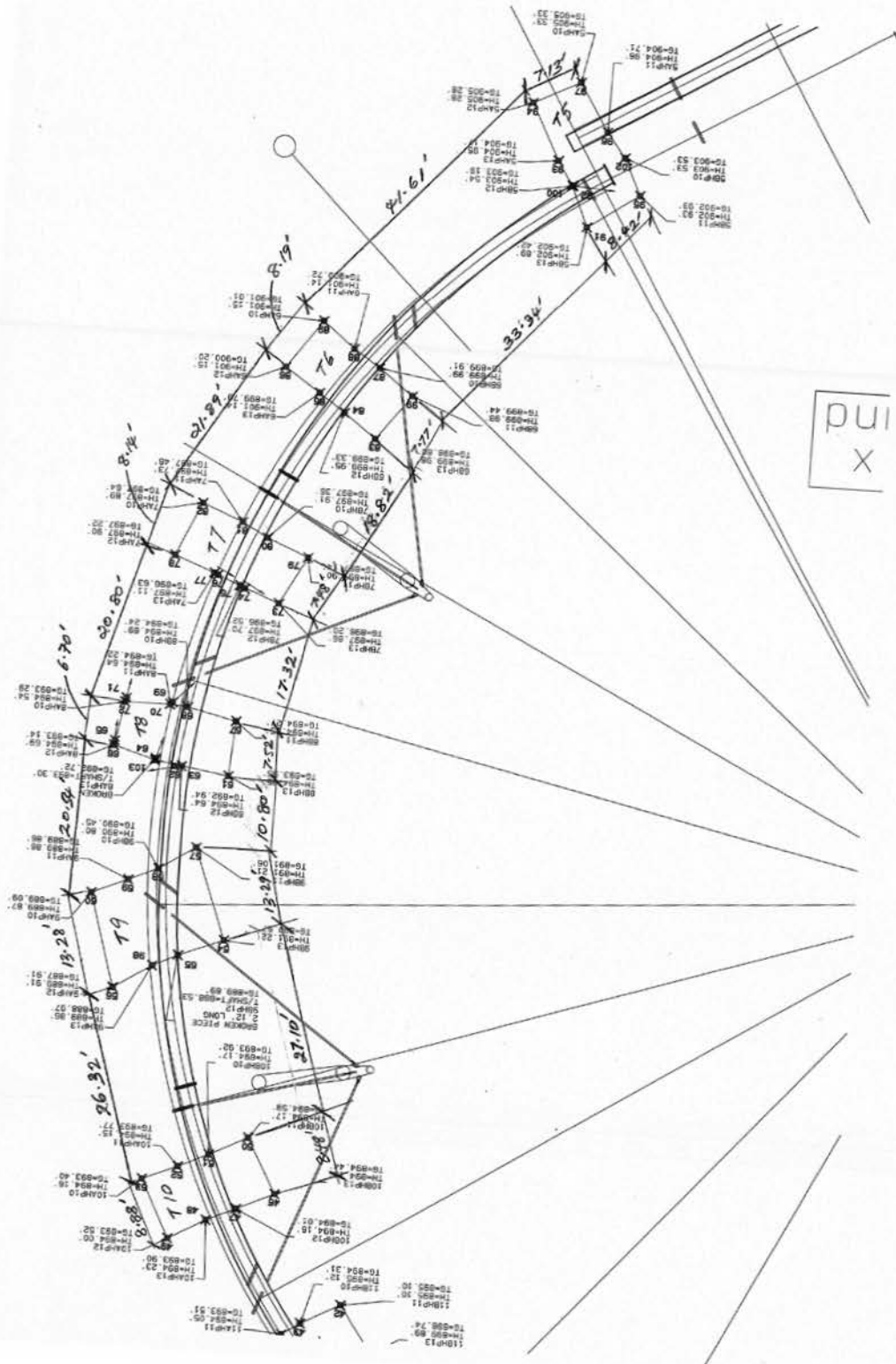
**Figure 27    Tower #7A after collapse**



**Figure 28    Tower #7B after collapse, No deformation**







<b>Tower 5</b>	97	96	94	93		102	95	100	91
	5AHP10	5AHP11	5AHP12	5AHP13		5BHP10	5BHP11	5BHP12	5BHP13
Top of Helix	905.33	904.96	905.28	904.95		903.53	902.93	903.54	902.89
Top of Grade	905.33	904.71	905.28	904.17		903.53	902.93	903.19	902.42
Ht. above ground (in)	0.0	3.0	0.0	9.4		0.0	0.0	4.2	5.6
Min. Embedment (ft)	7.0	6.8	7.0	6.2		7.0	7.0	6.7	6.5

<b>Tower 6</b>	89	88	86	85		87	99	84	83
	6AHP10	6AHP11	6AHP12	6AHP13		6BHP10	6BHP11	6BHP12	6BHP13
Top of Helix	901.15	901.14	901.15	901.14		899.99	899.98	899.95	899.98
Top of Grade	901.01	900.72	900.12	899.79		899.91	899.44	899.33	898.8
Ht. above ground (in)	1.7	5.0	12.4	16.2		1.0	6.5	7.4	14.2
Min. Embedment (ft)	6.9	6.6	6.0	5.6		6.9	6.5	6.4	5.8

<b>Tower 7</b>	82	81	78	77		80	79	74	73
	7AHP10	7AHP11	7AHP12	7AHP13		7BHP10	7BHP11	7BHP12	7BHP13
Top of Helix	897.89	897.73	897.9	897.11		897.91	897.9	897.7	897.86
Top of Grade	897.64	897.48	897.22	896.63		897.36	897.14	896.52	896.2
Ht. above ground (in)	3.0	3.0	8.2	5.8		6.6	9.1	14.2	19.9
Min. Embedment (ft)	6.8	6.8	2.9	2.4		6.5	6.2	5.8	5.3
Min. Embedment (in)	80	80	35	29		77	74	68	63

<b>Tower 8</b>	72	70	66	64		68	67	63	61
	8AHP10	8AHP11	8AHP12	8AHP13		8BHP10	8BHP11	8BHP12	8BHP13
Top of Helix	894.54	894.64	894.69	893.3		894.69	894.85	894.64	894.85
Top of Grade	893.29	894.22	893.14	892.72		894.24	894.27	892.94	893.35
Ht. above ground (in)	15.0	5.0	18.6	7.0		5.4	7.0	20.4	18.0
Min. Embedment (ft)	1.8	3.6	1.8	6.4		1.4	6.4	2.6	5.5
Min. Embedment (in)	22	43	22	76		17	77	31	65

<b>Tower 9</b>	60	59	56	98		58	57	55	54
	9AHP10	9AHP11	9AHP12	9AHP13		9BHP10	9BHP11	9BHP12	9BHP13
Top of Helix	889.87	889.86	889.91	889.86		890.8	891.21	888.53	891.22
Top of Grade	889.09	889.86	887.91	888.97		890.45	891.06	889.69	890.67
Ht. above ground (in)	9.4	0.0	24.0	10.7		4.2	1.8	-13.9	6.6
Min. Embedment (ft)	6.2	7.0	5.0	6.1		6.7	6.8	Broken	6.4

<b>Tower 10</b>	53	52	49	48		51	50	47	46
	10AHP10	10AHP11	10AHP12	10AHP13		10BHP10	10BHP11	10BHP12	10BHP13
Top of Helix	894.16	894.15	894.00	894.23		894.17	894.17	894.16	894.19
Top of Grade	893.40	893.77	893.52	893.90		893.92	894.59	894.01	894.44
Ht. above ground (in)	9.1	4.6	5.8	4.0		3.0	-5.0	1.8	-3.0
Min. Embedment (ft)	6.2	6.6	6.5	6.7		6.8	Broken	6.9	7.3

**Figure 31 Helical Anchor Embedment**



**Figure 32 Helical Anchors at T7 and T8 before cutting**





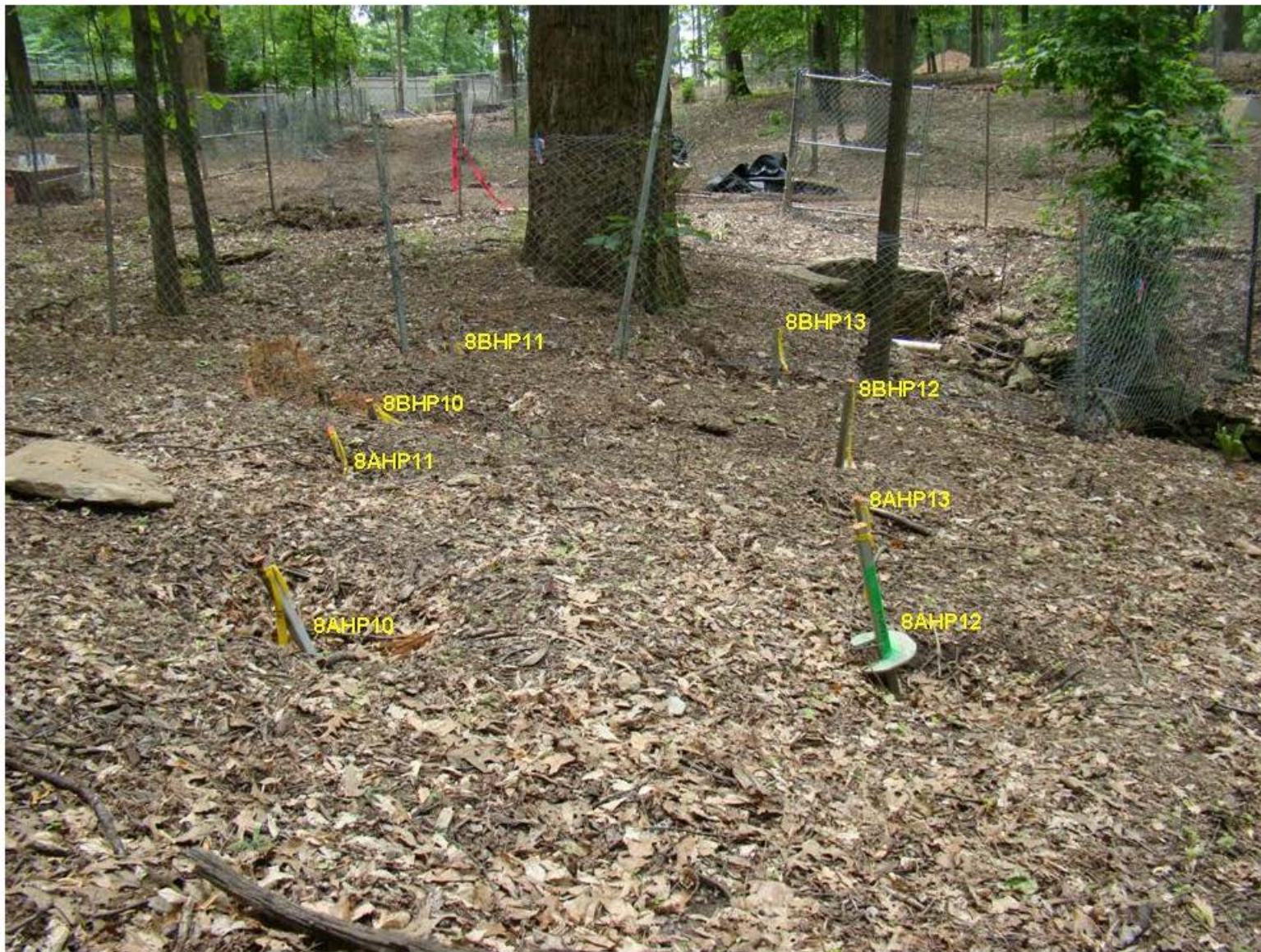
**Figure 33 Helical Anchors at T7 and T8 before cutting**





**Figure 34 Helical Anchors at T7 taken after collapse**





**Figure 35 Helical Anchors at T8 taken after collapse**

W10x33				
<i>Fy</i> (average )	<i>z<sub>x</sub></i>	<i>M<sub>p</sub></i>	Failure Load <b>P<sub>max</sub></b>	
<i>ksi</i>	<i>in<sup>3</sup></i>	<i>k-ft</i>	<i>kips</i>	
As Designed	50.0	38.8	161.7	64.7
Tested				
LL= 25psf, Eff. LL factor = 1.368, phi = 0.9				
As Designed				42.5
Tested				
LL = 19psf, Eff. LL factor= 1.365, phi = 0.9				
As Designed				42.6
Tested				

2-W10x19			
<i>Fy</i> (average )	<i>z<sub>x</sub></i> (average )	<i>M<sub>p</sub></i>	Failure Load <b>P<sub>max</sub></b>
<i>ksi</i>	<i>in<sup>3</sup></i>	<i>k-ft</i>	<i>kips</i>
50.0	43.2	180.0	72.0
59.3	43.04	212.7	85.1
(7SX09)			
			47.4
			56.0
			47.5
			56.1

2-W10x12			
<i>Fy</i> (average )	<i>z<sub>x</sub></i>	<i>M<sub>p</sub></i>	Failure Load <b>P<sub>max</sub></b>
<i>ksi</i>	<i>in<sup>3</sup></i>	<i>k-ft</i>	<i>kips</i>
50.0	25.2	105.0	42.0
47.0	24.62	96.4	38.6
(6SX03)			
			27.6
			25.4
			27.7
			25.4

1-W10x19 and 1-W10x12			
<i>Fy</i> (average )	<i>z<sub>x</sub></i> (average )	<i>M<sub>p</sub></i>	Failure Load <b>P<sub>max</sub></b>
<i>ksi</i>	<i>in<sup>3</sup></i>	<i>k-ft</i>	<i>kips</i>
50.0	21.6	90.0	36.0
50.0	12.6	52.5	21.0
Failure Load			57.0
59.3	21.52	106.3	42.5
47.0	12.31	48.2	19.3
Failure Load			61.8

AISC Table

1-W10x19

1-W10x12

As Tested

1-W10x19

1-W10x12

**Figure 36 Failure Load of Shoring Tower Beams (See Lab report attached)**

# **ATTACHMENT A**



May 29, 2009

Mr. Benjamin Ross  
Assistant Regional Administrator for Enforcement Programs  
U.S. Department of Labor – OSHA  
Atlanta Regional Office  
61 Forsyth Street SW, Room 6T50  
Atlanta, Georgia 30303

Phone: 404-562-2300  
email: [Ross.Benjamin@dol.gov](mailto:Ross.Benjamin@dol.gov)

**Subject: Report for Materials Testing Services  
W10 and W8 Steel shoring beams  
Atlanta Botanical Gardens Incident  
Atlanta, Georgia  
TEC Services Project No. TEC 09-0429.01**

Dear Mr. Ross:

Testing, Engineering and Consulting Services Inc. (TEC Services Inc.) is pleased to submit this report of our materials testing performed for the subject project. The purpose of our work was to perform chemical and physical materials testing of select shoring beams to determine yield and ultimate strength of the steel for compliance with ASTM A36 and/or ASTM A992 specifications. This report briefly presents background information and the results of our materials testing. Our services were performed in accordance with our OSHA contract.

## **BACKGROUND INFORMATION**

TEC Services was contacted by Michael Shea of OSHA regarding the subject project. We understand that OSHA requests materials testing of selected W8 and W10 steel beams that were used as shoring beams for the subject project. We understand that these beams were removed from the project site after a collapse during a concrete pour.

TEC Services was original contacted by Hardin representatives to perform selected testing. However, it was decided by the project team for TEC Services to perform the materials testing directly for OSHA. In an email on April 17, 2009, Mr. Shea and Mr. Mohammad Ayub of OSHA requested a specific scope of materials testing be performed on 6 selected beams. The purpose of this testing is to determine if the steel materials satisfy the ASTM A36 and/or ASTM A992 specification.

## **FIELD OBSERVATIONS**

On May 5, 2009 Mr. Philip Bender of TEC Services met with Mr. Marvin Sasser (Hardin Construction) and Mr. Americo Pagan (OSHA) at the Powder Springs warehouse to perform field measurements and obtain approximate 3 foot long samples from six selected W8 and W10 beams. Field measurements of the select beams were performed near the ends and middle of each member. Beam depth and flange width were obtained by steel rule and web and flange thickness measurements were obtained by ultrasonic thickness gauge. Field measurements were compared to AISC published values and ASTM A6-04. Some flange thickness dimensions were as much as 10% less than AISC published values. Our field dimensions are included in Table 1 attached.

Six approximate 3 foot long beam samples were removed and returned to our Lawrenceville, Georgia laboratory for further testing. Samples for testing were removed from beams labeled 6SX02, 6SX03, 6SX09, 7SX08, 7SX09, and 8SX03. A gas powered cut off saw was used to remove the samples from one end of each beam. Mr. Bender brought beam samples to the TEC Services Laboratory for testing.

## **LABORATORY MATERIALS TESTING**

Select material properties were tested and compared to ASTM specifications to determine the grade of steel used in the six selected W8 and W10 shoring beams. TEC Services performed tensile testing of the specimens to determine yield strength, ultimate strength and elongation of the material in substantial compliance with ASTM A370-05. IMR Test Labs performed chemical testing in accordance with ASTM A36-08. Per request, TEC Services also reviewed the results of the chemical testing and compared them to the chemical requirements for ASTM A992-06. The IMR Test Labs cover letter with chemical analysis test results is included as an attachment to this report.

The results of the chemical testing indicate the shoring beams tested (6SX02, 6SX03, 6SX09, 7SX08, 7SX09 and 8SX03) meet the requirements of ASTM A36 and ASTM A992 steel. The results of the physical tensile testing indicate the material meets the requirements of ASTM A36 steel (minimum yield strength of 36 ksi, ultimate strength 58 ksi to 80 ksi, and minimum

elongation of 21%). Two of the six beams tested (6SX03 and 8SX03) do not meet the physical requirements for ASTM A992 steel (yield 50 ksi to 65 ksi, minimum ultimate strength of 65 ksi, and minimum elongation of 21%). A summary of our physical testing results is included in Table 2 attached.

## CONCLUSIONS

Based on our physical and materials testing, the provided beams meet the requirements of ASTM A36 steel. Four of the six beams tested also meet the requirements of ASTM A992 steel (6SX02-W10x19, 6SX09-W10x12, 7SX08-W8x10 and 7SX09-W10x19). Two of the six beams do not meet the requirements of ASTM A992 (6SX03-W10x12 and 8SX03-W8x10). Some section flange thickness measurements were less than AISC published dimensions.

We appreciate the opportunity to provide our professional services to you and the management team on this important project. Should questions arise concerning this proposal or if we may be of further service please contact us at 770-995-8000.

Sincerely,

## TESTING, ENGINEERING AND CONSULTING SERVICES, INC.



Philip J. Bender  
Project Engineer



Kurt W. Heinrichs, P.E.  
Principal Engineer  
GA Registration No. 18741

Attachments: Table 1. Beam Measurements with Nominal Designation  
Table 2. Summary of Physical Test Results  
IMR Cover Letter with Chemical Analysis Results  
Instron Stress vs. Strain Curves