Commemorate the Legacy of Ralph B. Peck
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Seventh International Conference on Case Histories in Geotechnical Engineering
and Symposium in Honor of Clyde Baker

SLURRY CAISSON PROBLEMS AND CORRECTION IN CHICAGO

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ABSTRACT

The paper describes the design and construction history of 67-story and 45-story residential towers in Chicago which were constructed on straight-shaft caissons supported on the surface of dolomite bedrock at a design bearing pressure of 90 tons per square foot (tsf). The use of the 90 tsf bearing pressure was a first in Chicago and a strong departure from the Chicago code method of requiring rock sockets at least one to six feet deep along with permanent steel casing. The caissons were constructed by using polymer drilling slurry and tremie concrete pouring procedures.

This paper presents a brief history and evolution of the Chicago caisson to provide context to the project design and describes the load testing program used to prove the design and performance of the foundations. The non-destructive testing and coring programs used to check the concrete quality identified defects in several shafts which required remediation. The methods used to remediate the defective shafts included pressure grouting, shaft replacement, and large-strain dynamic load testing.

INTRODUCTION

It is fitting that this symposium is being held in Chicago to celebrate the careers of Ralph Peck and Clyde Baker, two geotechnical engineers who have influenced foundation design in the City more than any others. The history of high rise building foundation design and construction in Chicago prior to World War II is described by Peck (1948) and after that by Baker, et al (1984). The transition in Chicago from shallow foundations to deep foundations and the evolution of the Chicago caisson are well described in these documents and are must-reading for any geotechnical engineer working in Chicago.

Perhaps the best example of the transition from shallow to deep caisson foundations occurred for the construction of the current Chicago City Hall. Fig. 1 shows the then-current City Hall being demolished in 1908 after only 23 years in service. The reason for the demolition was 14 inches of differential settlement experienced by its shallow foundations even though the structure was only five stories tall. Figure 2 shows the demolition progressed to the basement level where the massive stepped footings are revealed. This photo also shows the excavation of the new "caisson wells" at the far end of the site as evidenced by the five A-frames covered with white tarps. The new City Hall foundations were hand dug to the top of rock using vertical wood sheeting and thin steel compression rings as shoring to keep the excavations open, not air pressure as in a true caisson. Figure 3 shows one of the 9.5 ft diameter caissons after excavation and prior to concreting.

The term caisson was used in the early 1900s due to the similarity in construction with caissons of the time which were hand dug under air pressure below major bridge structures. Today, the term caisson is largely a local term but is synonymous with drilled shaft, drilled pier, bored pile or cast-in-drilled-hole piles which are the common terms used in different parts of the world.

FIRST ROCK BEARING CAISSON DESIGN

The top-of-rock caisson design was typical for this time period. The actual design bearing pressure is uncertain but if we assume a 2000 kip column load for the 10-story heavy masonry structure the caisson in Fig. 3 would have had a design bearing pressure of about 15 tons per square foot (tsf). Another estimate can be based on the typical concrete strength of the time period which was around 1800 psi. Assuming a concrete factor of safety of 4.0 would result in a maximum rock bearing pressure of about 30 tsf. Thus, it is likely the actual value was between these two numbers. Because of the use of the wood lagging, any contribution from side friction was ignored.

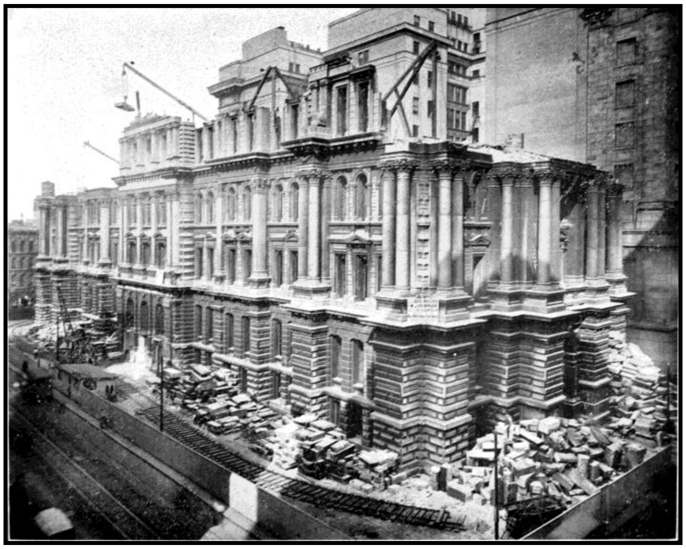

Fig. 1. Demolition of the third City Hall due to excessive settlement in 1908 after only 23 years of service.

Fig. 2. Demolition of the old City Hall in 1908 showing the massive stepped footings (left foreground) being replaced by hand-dug caissons (five white-topped A-frames at the top of the photo) for the new structure.

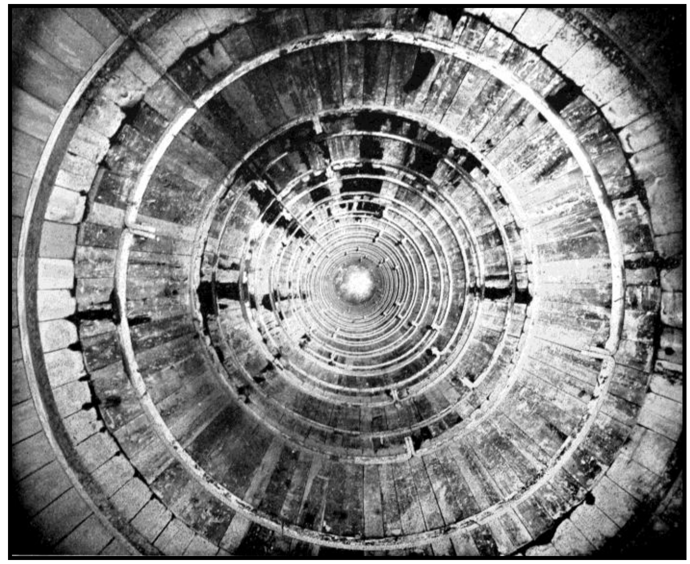

Fig. 3. View down a 100 foot deep 9.5 ft diameter hand-dug Chicago caisson for City Hall in 1909.

CHICAGO GEOLOGY

The impetus for the development of deep foundations was the ever increasing desire to build taller buildings coupled with the reality of Chicago geology. Figure 4, excerpted from Peck and Reed, 1954, shows that Chicago is founded on five glacial till sheets overlying dolomite bedrock which is typically 90 to 140 feet below grade. Chicago dolomite is a hard rock with typical unconfined compressive strengths in the range of 7,500 to 15,000 psi.

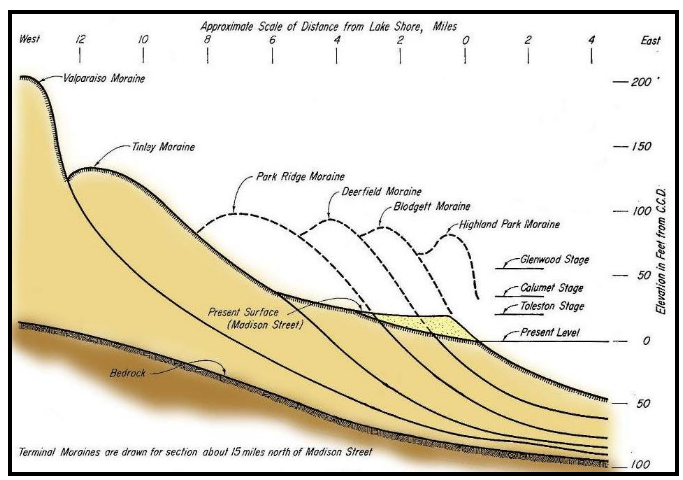

Fig. 4. Chicago geology consists of successively older and denser glacial till sheets with depth as depicted in Peck's 1954 classic paper.

The shallowest and youngest till sheet, the Blodgett was formed under water at the end of the ice age and was not compressed by subsequent glaciers. This material is soft to very soft clay with water contents often approaching or exceeding 40 percent, is often 40 ft thick, and was responsible for the excessive settlement of City Hall's shallow footings. Due to the soft clay, the majority of neighborhood buildings in Chicago are 3 to 4 stories in height – the maximum height that can be supported on shallow footings without excessive settlement.

The second and third till sheets, the Deerfield and Park Ridge, represent typically stiff to very stiff clay which increase in strength and decrease in water content with depth. The fourth till sheet, the Tinley generally consists of very hard silty clay or clayey silt with some sand and gravel. This till sheet represents the soil that is commonly referred to as Chicago "hardpan" and also provides the bearing for a majority of Chicago high rises at bearing pressures up to 25 tsf. The deepest and oldest till sheet often found above bedrock, the Valparaiso, typically consists of very dense saturated sand, gravel and silt which are under a water head.

CHICAGO CAISSON EVOLUTION

Because of the difficulty (and danger) in hand digging through the granular, saturated Valparaiso till sheets (the timber pile supported landmark structure Orchestra hall settled 8 inches as a result of the hand mining of top-of-rock caissons for the adjacent Borg Warner Building in 1958), designers quickly experimented with stopping caissons on the Tinley hardpan. As described in D'Esposito 1924, a full scale load test was done to test the load bearing capacity of the Chicago hardpan at Union Station. Even though this test proved a bearing capacity of 87.5 tsf at 2.5 inches of settlement, the designers of Union Station used 6 tsf as the allowable bearing pressure and this became the accepted value in the Chicago code and is still in effect today. Today, perhaps 95 percent of all modern Chicago high rises up to 80 stories in height are supported on machine-dug, belled caissons on the Chicago hardpan.

For buildings over 80 stories in height, building loads and bell sizes become too large even for the Tinley hardpan. Thus, the tallest modern structures in Chicago (Willis Tower, Hancock Building, Aon Tower, and Trump Tower as examples) are supported on machine-dug, rock-socketed caissons. Chicago code allows a 100 tsf bearing pressure on caissons socketed at least one foot into sound rock. For each additional foot of penetration, the Code allows an additional 20 tsf bearing pressure up to a maximum of 200 tsf at six-foot penetration. Penetration into the sound rock requires coring equipment. Side friction in the rock socket is ignored.

Chicago code also requires that a full length, heavy wall permanent casing be socketed and grouted into the rock to obtain a seal. The Code does not allow the steel casing to contribute to the load capacity, though the confined concrete is allowed a higher concrete stress level. The code also requires that each caisson location be probed at least 8 ft below the bearing level to search for rock seams. These foundations are usually only considered for the tallest structures because of their great cost.

ONE MUSEUM PARK CASE HISTORY

The One Museum Park east and west condominium towers occupy one of the most dramatic locations in the Chicago skyline, framing the south end of Grant Park at the location of the former Illinois Central Train Station after which the area is named. The east and west towers are 67 and 45 stories tall, respectively and are of reinforced concrete construction. The towers are connected by a common 5-story podium which provides parking and amenities. The east tower was begun first with conventional bottom-up construction over a two-level basement. The west tower began construction shortly after the east tower, but with a five-level basement was constructed by top-down procedures. The maximum column loads in the east and west towers were 7000 and 5500 kips, respectively. The towers are shown in Fig. 5

Fig. 5. The One Museum Park west and east towers today.

Central Station Geology

The Central Station area geology is more complex than downtown Chicago. South of Roosevelt Road, the denser

Valparaiso and Tinley till sheets are typically absent or thin so that conventional belled caissons at bearing pressures up 25 tsf are not possible. Bearing for belled caissons is possible on the higher and weaker Park Ridge till sheet. This till sheet has water contents in the 20 percent range and unconfined compressive strengths in the 2 to 4 tsf range of a very stiff clay. The generalized soil profile at the Museum Park site is shown in Fig. 6. Based predominantly on the use of the pressuremeter tests, south-side Chicago high rises up to about 35 stories in height have been supported on the Park Ridge till at bearing pressures in the range of 9 to 12.5 tsf. Many of these high rises have caissons with bells as large as about 20 ft in diameter.

Another challenge in the area geology is that the Blodgett till sheet is softer than any other location in the City. The soft to very soft clay exhibits vane shear strengths as low as 350 psf and water contents approaching 45 percent. As a result, large caisson excavations have been known to squeeze shut if kept open for too long a period as discussed by Budiman and Kiefer, 2004. Because of the squeezing clay it is common to require temporary casing to depths of 50 ft to prevent off-site movements. As bells become larger and shafts exceed 5 ft in diameter the likelihood of squeeze increases.

Foundation Problem

Due to the geologic conditions alternative foundation types including rock-socketed caissons and driven piles were considered for Museum Park. However, while driven piles were possible for the east tower which was done with normal construction methods, they were not possible for the west tower which had a five level basement and was constructed with top–down procedures. Driven piles were also used on numerous towers in the Central Station area, but for this project their cost was considerably greater than the ultimate solution.

Similarly, the cost of the rock-socketed caissons was excessive because of the requirement for permanent steel casing and pre-coring at each caisson location. We estimated that the additional cost for the steel material alone would be on the order of two million dollars per tower.

Proposed Solution

The final solution proposed was a mixed foundation system consisting of belled caissons under the podium areas designed for 9 tsf and straight shaft caissons supported on the surface of bedrock for 90 tsf. The maximum bell size under the podium was on the order of only 10 ft which we felt could be constructed without significant squeeze or belling problems with open shafts and free-fall concrete. The use of straight shaft caissons for the towers constructed under polymer slurry reduced the need for temporary casing while solving the clay squeeze issue. Estimated settlement for the garage caissons was ¾ inch while the estimated maximum settlement for the top-of-rock caissons was ½ inch. Experience had shown that the settlement of the rock caissons was more related to the amount of sediment left in the base of the shaft rather than rock compression.

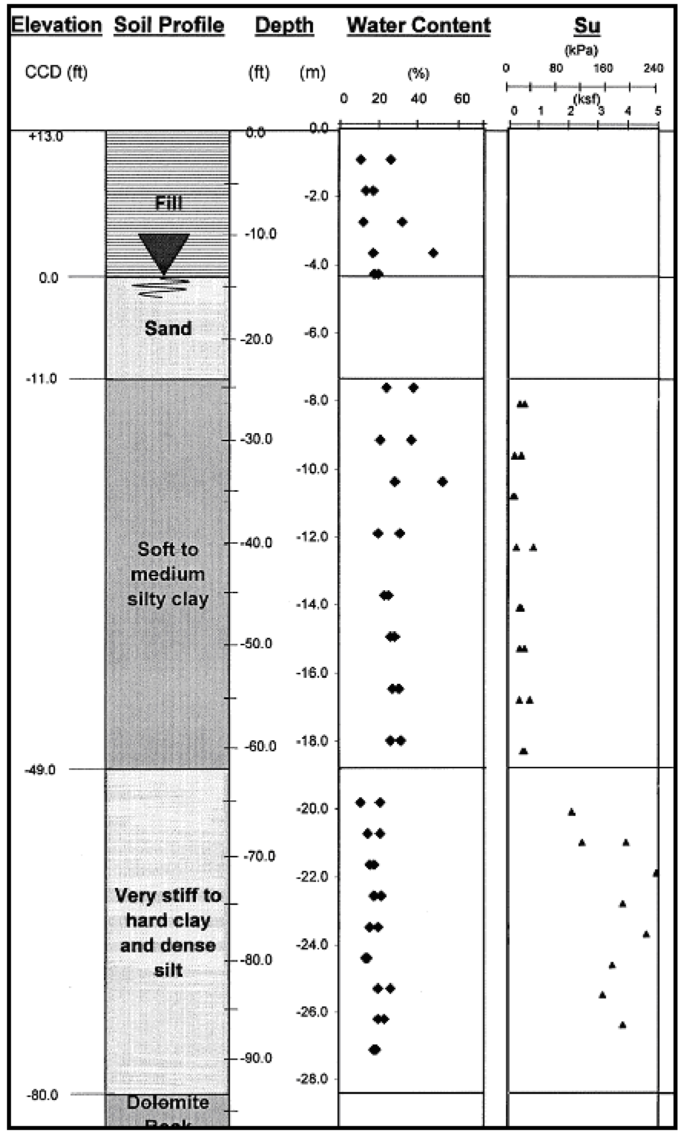

Fig. 6. Generalized soil profile and material index properties at the One Museum Park site.

Previous Top-of-Rock Slurry Caisson History

An important tenet in geotechnical engineering learned from Clyde Baker is to take small steps when increasing bearing pressures. The use of 90 tsf on the surface of rock was unprecedented in the City; however, four other recent major top-of-rock projects were completed previous to One Museum Park at bearing pressures varying from 45 to 75 tsf. These previous projects included the McCormick Place West Hall Expansion, 1845 S. Michigan, 16th and Prairie and Museum

Park Tower 4.

The McCormick Place Expansion project was the first project in the City to use the bi-directional load cell method on one of four load tests. The successful use of this method opened the door in the City to accepting the test method with Quick Test procedures, rather than requiring 48 hour tests. The tests proved a top-of-rock bearing pressure of 75 tsf for a portion of the structure. At this project, the shafts were drilled dry because clay extended to the rock surface which varied from about 40 to 70 feet in depth.

1845 S. Michigan was the first polymer drilling caisson project in the City. The design here was for 45 tsf on the top of rock which was considered to be conservative and was based on pressuremeter tests. However, during construction, some of the central shafts were placed on boulders or shelf rock as shown by planned post construction coring which resulted in some shafts needing remediation. Lessons learned on this project were to increase the number of borings to check rock surface irregularities and to increase rock surface grinding time as described in Baker and Kiefer, et al 2004.

Within Central Station and Museum Park, two projects were used as the first locations where O-cell tests were performed in production caissons to prove 75 tsf bearing pressures. Lessons learned at these projects included using a high factor of safety on concrete stress level (8000 psi concrete for a 75 tsf bearing pressure) to account for possible concrete problems. At 16th and Prairie, random cores found weaker concrete just above rock, probably resulting from mixing of sediment with the tremie concrete; however, the compressive strengths still exceeded 4000 psi thus, the concrete factor of safety did not drop below 4.0 as required by Chicago code. At Museum Park Tower 4, the load test was taken high enough to prove 90 tsf on rock at a factor of safety of 3.0, even though the design was 75 tsf. This project was immediately adjacent to the One Museum Park project and had already proven in effect that 90 tsf was reasonable.

The gradual progression of the results from these load tests on Chicago's south side effectively paved the way with City regulators for the proposed 90 tsf design at One Museum Park.

Exploration and Testing Plan

Although there was some precedent for placing up to 30-story buildings on top-of-rock caissons, the high bearing pressure and lack of specific code required a comprehensive testing and exploration program. This program included:

- About 10 rock cores per tower to map the rock surface (about one core for every four caissons).
- Two bi-directional Osterberg load tests on the first two production caissons to prove the bearing pressure and settlement of the foundations.
- Grouting of the O-cells and coring of the lower 10 ft of the production caissons.
- Non-destructive testing of all caissons poured under slurry.
- Use of design concrete stress level less than 0.15 f'c.
- Full length cores of three caissons per tower.

The quality of the rock at the Museum Park site was good. The top of rock varied by a maximum of about 2 feet across the site and recoveries typically exceeded 95 percent with RQD exceeding 75 percent. Relatively little fractured rock was encountered at the site.

Bi-directional Load Test Design. Since there was little doubt in our minds that the dolomite rock could support a 90 tsf design stress, the real purpose of the load testing was to test the contractor's ability to excavate, clean and construct the shafts to achieve the expected performance. The second purpose was to correlate the inspector's feel of the bottom cleanliness and rock hardness for a given amount of grinding time to the settlement performance. Thus, if the construction and clean-up procedures resulted in successful test shafts, these procedures would become the minimum standard for the remaining production caissons. The third purpose was to prove the design and performance to the City.

The challenging part of performing two full scale load tests was to do them quickly and economically. This was achieved by performing the load tests on production caissons using the bi-directional load test method using the Osterberg load cell. This method eliminated the need for massive weights, reaction beams or rock tie down anchors. In this method, (and since our goal was to measure the end bearing) the O-cell was located on the rock surface to maximize the load transferred directly into end bearing while providing the weight and side shear resistance of the entire shaft as a reaction. A schematic of the test configuration is shown in Fig. 7. At first glance, this might seem to be impossible given that Chicago code does not allow for any side friction on shafts because of the thick deposit of soft Chicago clay. However, previous load tests had shown that friction is developed in the clay and the approximate penetration through 30 ft of very stiff to hard till should generate in excess of 2000 kips of reaction force for the 6.5 foot diameter test shafts. This load while significant would be able to generate only a 35 tsf end bearing pressure over the entire shaft base. To remedy this situation, we opted for a limited base area test and placed the O-cell on a two-foot diameter bearing plate as shown in Fig. 8. Thus, the tests were essentially massive plate load tests designed to measure unit end bearing resistance.

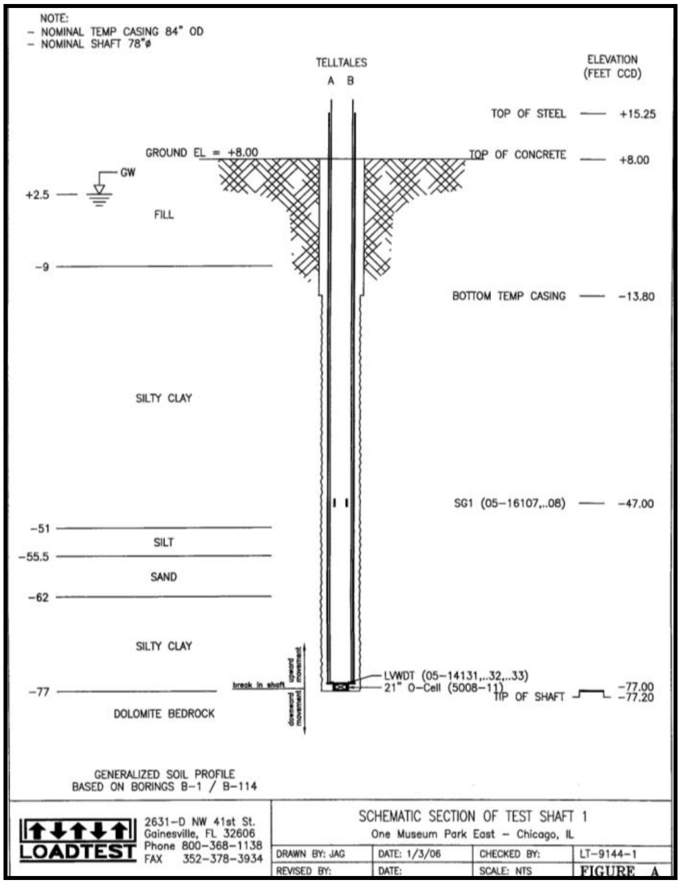

Fig. 7. Schematic of limited base area Osterberg load test configuration on Test Shaft 1, a production caisson.

Fig.8. Cage and base plate configuration for a limited base area Osterberg load test on a production caisson.

Load Test Results. The load test results from the two test production shafts are shown in Fig. 9 and Fig. 10. These results show that from 1700 to 2000 kips in load was transferred to the base plates resting on the rock surface which proved end bearing pressures of 275 to 320 tsf. Thus, even with the measured friction ignored completely, the end bearing resistance of 90 tsf was proven to a factor of safety of 3.1 to 3.6.

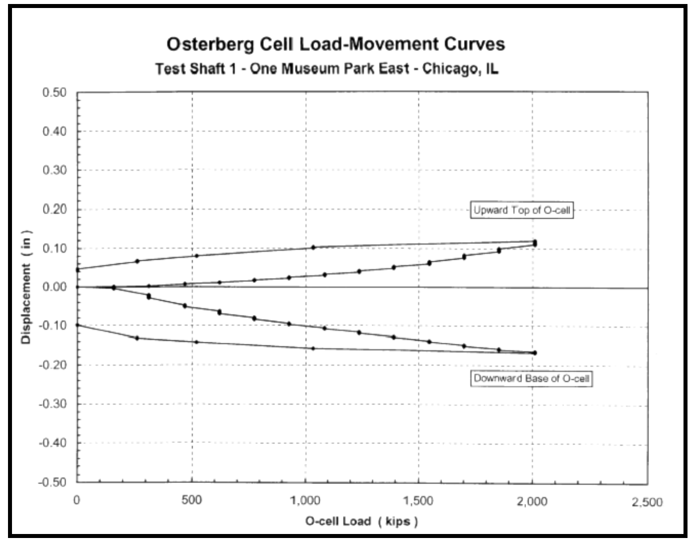

Fig. 9. Load test results for Test Shaft No. 1 at One Museum Park East.

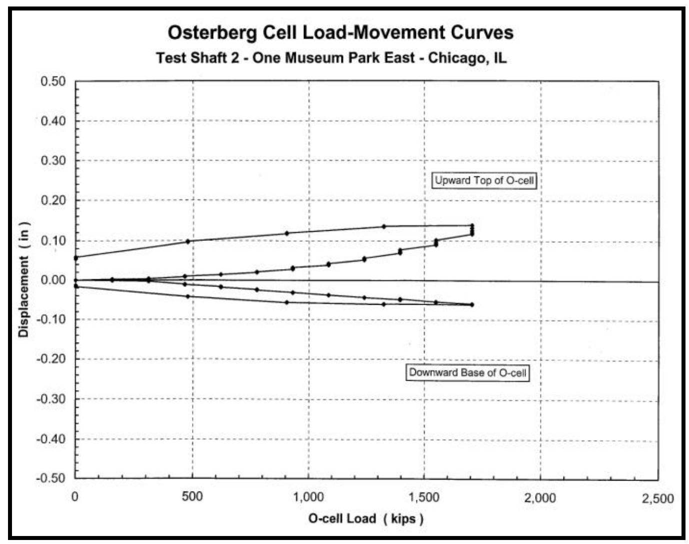

Fig. 10. Load test results for Test Shaft No. 2 at One Museum Park East.

It is worth noting that while side friction is ignored within the Chicago code, the actual side friction measured within the soft Chicago clay and Park Ridge till were 200 psf and 2250 psf, respectively at movements of less than 1/8 inch. The average value measured over the entire length of the 85 foot long shafts was 950 psf. These values were not ultimate values, but using some judgment of the shape of the load movement curve, we estimate that the ultimate values would have been perhaps 50 percent higher. NAVFAC DM 7.1 recommends adhesion factors less than 0.4 in very stiff clays; however, the results of the load tests justify an adhesion value of at least 0.65 based on the average shear strength of 3500 psf measured in unconfined compression tests on Shelby tube samples. If the load tests had been taken to failure, the likely adhesion value would have been unity (1.0).

The reserve capacity in the O-cells used for this project would have allowed the tests to continue to twice the actual load applied. The tests were stopped however, because there was no desire to fail the shafts in side shear and push the shafts up more than ¼ inch away from the bedrock. Thus, even though the interior of the O-cells and base of the shafts were pressure grouted with neat cement grout, even if the grouting was not done or was not successful, the maximum downward movement of the shaft under building load would have been limited to ¼ inch plus rock compression, which would have been acceptable. After grouting, the bottom ten feet of each test shaft was cored though a pre-placed PVC access tube to check the concrete, grout and rock interface.

Construction Monitoring Plan

Experience at many previous projects had shown that proper tremie pouring procedures, bottom clean-up and verifying that the caissons were properly situated on the sound rock surface were more critical than the somewhat arbitrary selection of the design bearing pressure. While an experienced contractor and inspector were critical, Clyde Baker always recommended “trust, but verify.”

The production caisson load tests were completed using a minimum grinding time of 30 minutes. Over the course of the project, this grinding time was increased to 1 to 2 hours to achieve a flat bottom. We required the use of a flat bottom rock auger with carbide teeth which were checked and changed regularly. A central “stinger” was not allowed because of the concern that the auger would simply ride on a dull stinger rather than grinding the rock surface. A key indicator in the field was to observe the Kelly bar to check that it did not ride up and down as the auger was turned. Riding up and down was an indicator of an uneven bottom, boulder or possible shelf rock.

While it is common in some state DOT specifications to require the use of a Shaft Inspection Device such as a SID or mini-SID, this was not the practice employed in Chicago. We found that using a weighted rod as a hard rock sounder (Fig. 11) was sufficient to sound the rock and detect the thickness of sediment left on the bottom. The rock sounder was used at the center of the shaft and the four compass points to check cleanliness. The weight of the probe in conjunction with the thin point was such that the probe would stick, even in hard clay, but would bounce when struck on hard rock. If more than 1 inch of sediment remained on the bottom (the length of the probe tip), this was also felt as sponginess in the response. Deeper sediment would also accumulate on the top of the lip of the probe. As many as 15 to 20 passes with a carbide bladed, flat bottom muck bucket was needed to remove sediment.

Fig. 11. A hard rock tester similar to the design used on the OMP project.

The most important pouring procedure was to use a proper separator “pig” between the slurry and concrete in the tremie pipe and to always keep the tremie embedded at least 5 feet into the concrete once the pour had started. Vermiculite was used as the separator for this project. Polymer slurry was maintained at a Marsh funnel viscosity of between 75 and 80 seconds. After cleaning, each shaft was left for a minimum of two hours to allow fines in suspension to settle out of the slurry. A final clean-up pass was then done and the slurry was checked for sand content just before concreting. A sand content of less than 1 percent was required and typical results were less than 0.5 percent. After the cage was inserted, a final bottom sounding was performed to check that sediment or material from the shaft walls had not caved to the bottom.

Non-Destructive Testing. For the east tower, Impulse Response Spectrum (IRS) tests were performed on each rock caisson. The test consisted of tapping the top of the shaft with an instrumented hammer. A geophone recorded the wave reflection and provided information on the depth to possible anomalies or the shaft base. For the west tower, 15 caissons under the core mat which were constructed within a circular cofferdam were also tested by the IRS method, while the remaining rock caissons were cast with four steel access tubes for Cross-hole Sonic Logging (CSL). The CSL testing was used due to the limited access caused by the deep cut-offs and top-down construction method. The CSL method used a source and receiver lowered into two tubes. The device measured the transit time of a wave pulse between the two tubes. Tests were done in all tube combinations every few inches in depth so that two profiles across the heart of the caisson and four profiles around the perimeter were recorded.

A minimum of three full length concrete cores were planned for each tower so that caissons with detected anomalies or defects could be checked. If field observations or NDT testing did not indicate possible problem caissons, the cores would be performed randomly.

Coring and Grouting Caisson 90

IRS testing in the east tower revealed three caissons with anomalies. Two of these were cored with no defects being found. In one of the shafts, the only difference in concrete detected was some slight segregation of concrete as evidenced by a lack of large aggregate. The compressive strength of this zone exceeded the design requirement and was equal to the strength of the core above and below the anomalous zone. The density of the anomaly was about 5 pcf less than the remaining concrete. This indicated the sensitivity of the testing procedure, but also made it clear that a minor anomaly did not equate to a defect.

At Caisson 90 an anomaly was detected at about 20 ft below the top of the caisson. Three cores were advanced to 30 ft, but two of the cores showed good concrete. The third core revealed a zone of broken concrete from 22 to 24 feet as shown in Fig. 12. This was unusual in that the concrete appeared to be unsegregated and hard but was nevertheless shattered. The compressive strength of the core above and below the shattered zone was in excess of 9600 psi.

Fig. 12. Broken concrete core from approximately 22 to 24 feet at Caisson 90.

Caisson 90 was a podium caisson that was converted from a belled caisson to a rock caisson due to water problems. The shaft construction record also indicated that a “mudslide” had occurred at a depth of 15 ft while concreting. As a podium foundation, the caisson was not heavily loaded and could easily support the design load even for a worst case assumption that 1/3 the shaft was bad. Despite, this, the shaft was remediated by pressure grouting with 8000 psi neat cement grout as shown in Fig. 13. The grouting could not achieve a high pressure in the bad core hole even though several cubic feet of grout was pumped. We assumed that the grout was exiting the caisson and simply filling the loose fill and soft clay at the level of the anomaly. Subsequent IRS testing after remediation confirmed that the anomaly was gone.

Fig. 13. Remediation pressure grouting of the shallow anomaly at Caisson 90.

Coring, Grouting and Reinforcing Caisson 13

CSL testing at Caisson 13 in the west tower revealed a complete loss of signal in all tube pairs in the bottom 10 ft of the shaft. When an anomaly occurs over all of the tube pairs it is usually an indication of a serious defect. For this shaft, the field construction records provided no indication of a problem. Three cores in this shaft showed segregated and weak concrete in two of the three cores. Though the concrete strengths were considerably less than the nominal 10,000 psi design strength, they were close to the 5000 psi compressive strength needed for a factor of safety of 4.0 on the concrete. Despite the zones of poor concrete, the interface between the concrete and rock was excellent as shown in Fig. 14. At this shaft, side friction as proven in the load tests was considered to estimate the net design stress at depth which reduced concern for this shaft further. Despite the acceptable stress level, this shaft was also pressure grouted through the core holes and 20-foot lengths of 150 ksi, #14 bars were placed in the grout holes to further reinforce the shaft.

Fig. 14. Bottom of concrete core at Caisson 13 showing clean contact between concrete and dolomite bedrock at 91.8 ft.

Replacement of Caisson 2

During installation of Caisson 2 at the west tower, our field technician noted that the tremie pour was interrupted and the tremie pipe was pulled out of the concrete twice and reinserted because it plugged. We recommended mucking out the shaft and starting over, but the contractor declined to do so, hoping that the CSL testing results would show that the caisson was good. The CSL results in Fig. 15 show that the concrete was of very poor quality throughout the majority of the shaft. A full length core found weak segregated concrete, washed-out gravel and missing concrete as shown in Fig. 16. Compressive strength tests on intact portions of the core showed that the entire shaft was compromised with compressive strengths between 1700 and 4200 psi. Because of these results, replacement of the shaft was required.

Replacement options included two new shafts connected by a grade beam and complete replacement of the shaft. The contractor elected to replace the shaft by drilling out the upper 20 feet and coring the lower 30 ft of the shaft. Thus, a new shaft, 3.5 feet in diameter was cored inside the existing 4.5-foot shaft. This avoided the difficulty of removing the rebar cage and effectively used the shell of the existing shaft as a permanent casing. To make this work, the new shaft was extended one foot into sound rock by coring below the level of the original caisson to increase the allowable bearing pressure to 100 tsf based on Chicago code. A new cage with CSL tubes was cast into the replacement shaft and testing confirmed the integrity of the new concrete.

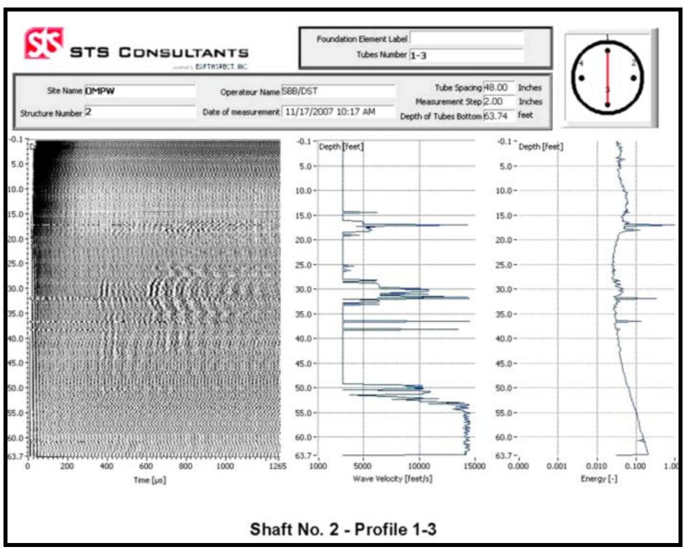

Fig. 15. Cross-hole Sonic Logging results at Caisson 2 showing very low wave velocity (less than 5000 ft/sec) throughout most of the shaft.

Fig. 16. Concrete core at Caisson 2 showing weak concrete, segregated concrete and gravel zones above bedrock from 84 to 93.5 ft.

STS Consultants Ltd.
Consulting Engineers

Compressive Strength of Concrete Cores
ASTM C-42

Laboratory Services Group *750 Corporate Woods Parkway Vernon Hills, Il 60061* *Phone: (847) 279-2500 Fax: (847) 279-2550*

STS Project No.: **200701642** **12/13/07**
Project Name: **One Museum Park West Caisson C-2**

Date Sampled: **11/20/07** Test Condition: As Received
Date Received: **11/26/07** Mix Design Strength: 10,000p.s.i @ 56 days
Date Tested: **11/27/07**
Date Cast: **11/6/07**

Summary of Test Results

Core No.	Ht. (in.)	Cap Ht. (in.)	Dia. (in.)	Area $(in)^2$	H/D Ratio	Corr. Factor	Load (lb)	Strength (psi)	Weight (gm.)	Unit Wt. (pcf) *	Location
1	4.74	4.91	2.49	4.85	1.97	0.9976	12340	2,540	712.4	118.0	38'
2	4.59	4.74	2.49	4.85	1.91	0.9928	8700	1,780	754.8	129.3	41.5'
3	4.69	4.89	2.49	4.87	1.96	0.9968	11590	2,370	818.4	136.5	43.5'
4	4.59	4.77	2.49	4.86	1.92	0.9936	9680	1,980	848.5	144.8	62.5'
5	4.67	4.90	2.48	4.83	1.98	0.9984	10440	2,160	892.9	150.9	72'
6	4.72	4.87	2.49	4.88	1.95	0.9960	20560	4,200	919.9	152.0	92'

Fig. 17. Compressive strength test results of Caisson 2 showing 21-day compressive strengths of only 1800 to 4200 psi and unit weights as low as 118 pounds per cubic foot for 10,000 psi design strength concrete.

Dynamic Load Test on Caisson 69

Random concrete cores were planned in both towers to check concrete quality. At the location of Caisson 69 in the east tower, the core encountered about 4 inches of soil between the concrete and rock surface as shown in Fig. 18. Review of the IRS testing at the shaft showed that the bottom reflection could be considered "soft" in comparison to the other shafts tested. Nothing inordinary was noted in the as-built log by the inspector. It appeared that the soil zone was compressed silt and clay. If this was an isolated zone from a clay lump which fell into the shaft prior to concreting it might not be a concern. However, if the zone extended across the entire base, it appeared that more than 2 inches of settlement could occur before the caisson "fetched up" on the rock. To check this, three more cores were attempted, but all of them drifted out of the shaft before reaching the base of the shaft. With no other options, we chose to perform a full scale dynamic load test on the shaft. Because of the relatively rapid performance of this test, we were able to also perform a dynamic load test on Caisson 79, a good shaft with a hard bottom that was also cored. By testing both shafts, a comparison between the two could be made to help interpret the results.

Fig. 18. Core at Caisson 69 showing 4-inch clay zone (top row) between concrete and dolomite bedrock at 84 ft.

To test the caissons, high strength cased caps had to be cast on top of the two shafts as shown in Fig. 19. To achieve a test load greater than the design loads which were about 5500 kips, it was necessary to use an Apple IV tester as provided by GRL. The test setup consisted of a 40 ton weight which was dropped from a height of as much as 3 feet onto the shafts. The test load is about to be dropped on Caisson 79 in Fig. 20. We estimated that the test setup was capable of applying a 4000 ton test load which would allow the caissons to be tested to a factor of safety approaching 1.5 which would be acceptable for confirming the performance for a specific shaft.

Fig. 19. Top of Caisson 69 (and Caisson 79 in the background) prepared with a high-strength, cased concrete cap for dynamic load testing.

Fig. 20. Forty-ton weight about to be dropped on Caisson 79 to perform a 4000 ton dynamic load test.

The on-screen results from the load test on Caisson 79 are shown in Fig. 21. These results showed a load impact of about 7400 kips while monitoring the tensile stress in the shaft from the rebound which approached 600 psi. At Caisson 69, a total of four hits were performed which permanently pushed the caisson down ½ inch. Each successive hit was stiffer than the previous hit. The testing confirmed that the reaction of Caisson 69 was about ½ the stiffness of Caisson 79 and confirmed that the clay layer appeared to extend over the entire shaft base. Based on the measured stiffness response from the dynamic load test, we estimated Caisson 69 was likely to settle an additional 1 inch as the building load was applied. While we felt this was acceptable, the structural engineer added a grade beam between Caisson 69 and two neighboring caissons to distribute the load and control the possible differential settlement.

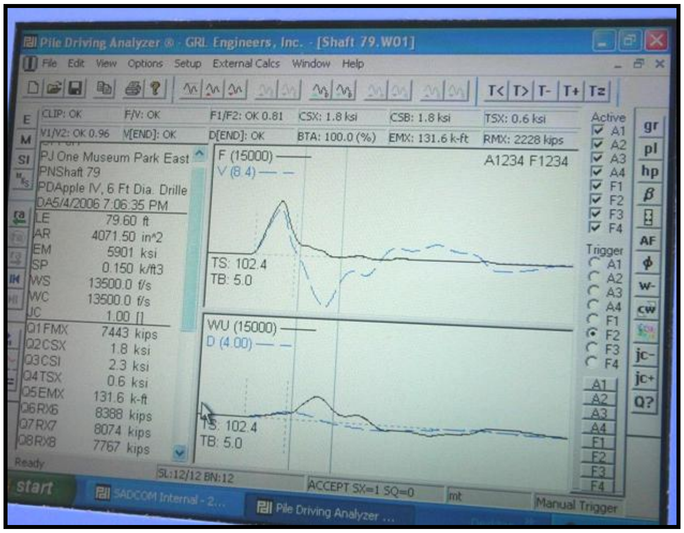

Fig. 21. In-the-field computer display of the response of a 4000 ton dynamic load test on Caisson 79.

CONCLUSIONS

1) Highly loaded, non-redundant, rock-supported end-bearing drilled shafts poured under slurry by tremie methods require greater care in design and construction to ensure concrete integrity than shafts in redundant groups, shafts designed for side friction, or shafts poured by free fall methods.
2) High safety factors (on the order of 7) and low concrete stress levels (0.15 f'c) are recommended for caisson concrete design where concrete is poured by tremie methods under drilling slurry for non-redundant, end bearing design.
3) An experienced, empowered, and knowledgeable testing agency technician (representative of the geotechnical engineer) is essential to check on proper bottom cleaning and proper concrete pouring procedures.
4) An experienced, conscientious drilled shaft contractor and foreman are essential.
5) Even with the most experienced contractor and inspector, tremie concrete pours must be verified by non-destructive testing because anomalies will occur.
6) Limited bearing area, bi-directional Osterberg load tests successfully proved rock bearing pressures in-excess of 270 tsf and allowed a design bearing pressure of 90 tsf at a factor of safety exceeding 3.0. on dolomite bedrock.
7) Full scale, large strain dynamic load tests to 4000 tons proved the load bearing capacity in excess of 140 tsf (Factor of Safety > 1.5).

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