Unexpected Caisson Problems, Soil Structure Interaction Predictions and Required Ground Modification

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ABSTRACT

The paper describes the caisson construction problems encountered and the required modification necessary for a 55-story residential high-rise in Chicago's near north side. Belled caissons were planned on a very thin hardpan bearing layer which was underlain by water bearing dense silt that extended to dolomite bedrock. Three filtered dewatering wells extending into the fractured rock surface were planned to reduce the hydrostatic pressure head within the silt to permit the belled construction.

A complete collapse of the dense silt layer during the installation of the first dewatering well undermined the planned belled caisson foundation system. An additional subsurface investigation, a compaction grouting program and further in-situ pressuremeter testing was then performed. Subsequent modified performance predictions required the addition of selective micropile underpinning after completion of the planned system of grade beams and belled caisson installation. Settlement monitoring during building construction confirmed settlements within or less than the predicted settlement range.

INTRODUCTION

The paper describes the caisson construction problems encountered and the required modification necessary for a 55-story residential high-rise in Chicago's near north side. Belled caissons were planned on a very thin hardpan bearing layer which was underlain by water bearing dense silt that extended to dolomite bedrock. Three filtered dewatering wells extending into the fractured rock surface were planned to reduce the hydrostatic pressure head within the silt to permit the belled construction.

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Project Design

The project consisted of a 55-story residential high-rise with a one level basement extending 15 feet below street grade. Tower column loads were in the 5,000 to 10,000 kip range with non-tower loads in the 1,500 to 5,000 kip range.

Site History and Site Investigation

The building site had previously been occupied by a 3-story building on footings and a 4-story building on small diameter belled caissons. A previous investigation was performed at the site for a proposed 9-story building. Information from a number of projects within one block of the site included a total of 45 soil borings that were extended at least to the hardpan layer with some extending to the deeper underlying dolomite. An additional 5 borings were performed at the site (1 before building demolition and 4 after). The anticipated soil profile based on all of the available boring information is shown in Fig. 1.

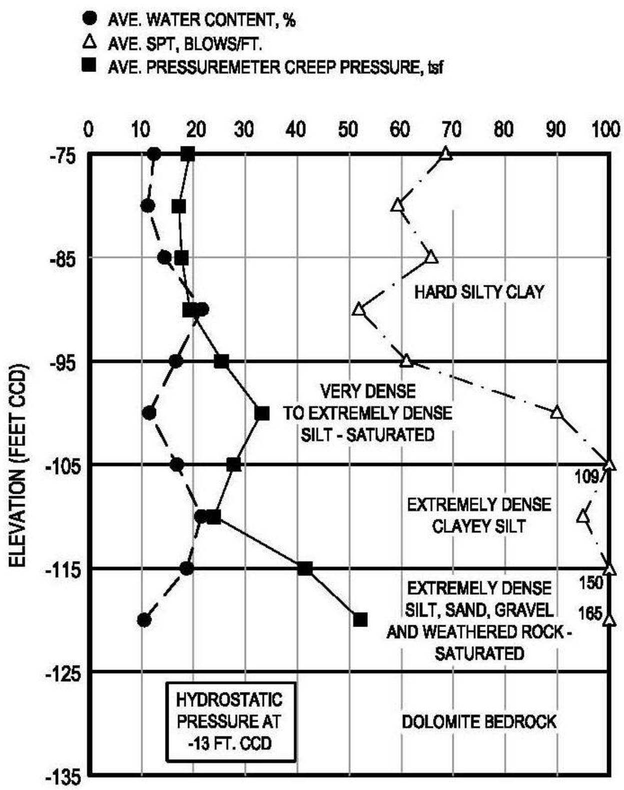


Fig. 1. Soil Profile based on Initial Subsurface Exploration

Based on experience, two potential problems were anticipated for the proposed caisson construction. One was the possibility of encountering water-bearing sand and silt at the surface of the anticipated hardpan bearing layer. The second was the presence of the existing caissons belled in the elevation range of -70 to -75 feet Chicago City Datum (CCD). The new caissons would need to extend deeper to fit below the existing caissons. Extending the caissons presented the risk that the belling operations could be de-stabilized by the hydrostatic pressure within the underlying silt. At a similar nearby project, filtered dewatering wells had been tried to lower the water head on the dense silt underlying the hardpan, with caissons designed to bear on the dense silt layer.

However, the dewatering wells were not successful, and the deep bearing solution had to be abandoned. The bells were constructed higher in the hardpan as close to the dense silt as possible without developing a blow-in potential and using a somewhat reduced bearing pressure based on the maximum sized bell that could be constructed within the old existing caissons footprint. The success of this solution influenced the design and construction of this new site.

Lessons learned on dewatering wells from the adjacent site were used on this project. Ultimately, satisfactory filtering of dewatering wells was achieved using a uniformly graded medium sandfill where the sand passes a No. 10 sieve and is retained on a No. 20 sieve. This was on the coarse end of a workable filter for the natural silt deposit. This would allow for some bleeding out of the silt, but because of the low permeability of the silt, it was not expected that much water would actually come out of the silt. Rather it would depend on water coming out of the cracks and fissures in the underlying dolomite.

<u>Design Testing Program</u>

The planned testing program at this site consisted of in-situ pressuremeter testing below the hardpan level in all five borings plus routine standard penetration, water content and strength tests.

The soil property information below -75 feet CCD at the time of design is summarized on Fig. 1.

<u>Foundation Design</u>

At the nearby project, the original intent of situating the caissons on the very dense silt proved impractical. Therefore the foundation design for this project anticipated situating the caissons within the hardpan, just above the very dense silt, but close enough to the dense silt that the silt properties governed design.

Three filtered dewatering wells were planned in the core area to reduce the potential for blow-in combined with the existence of the thin hardpan layer between the designed bell level and the very dense water bearing silt. Where it was possible to construct bells at the surface of the hardpan, the design bearing pressure was 36 ksf. Where it was necessary to extend the bells deeper and closer to the dense silt layer in order to get below existing caissons, the design bearing pressure was 48 ksf. This approach is similar to what was accomplished on the nearby problem caisson project referred to where it had become necessary to pull the bells up above the dense silt due to insufficient water head lowering. Settlement predictions varied from ½ inch for the smaller bells supported on the 36 ksf hardpan material in the depth range of -75 to -85 feet CCD to ¾ inch for the larger bells designed just above the dense silt at 48 ksf in the elevation range of -90 to -95 feet CCD.

<u>Construction History</u>

Because of the presence of existing caissons at the site, it was necessary to design the lowest level of core walls as minimum 22 foot deep grade beams throughout the core to tie the caissons together and facilitate load sharing where caissons could not be located in ideal locations under concentrated areas of load. No architectural openings were allowed in the basement level of core walls to allow the walls to act as grade beams without the additional excavation required to place a level of grade beams below the basement level.

Construction of caissons with bells upon the hardpan at 36 ksf was completed before dewatering well installation since lowering of the water table was not considered necessary. Unfortunately, construction problems developed during the installation of the first dewatering well (Well No. 3). The 4-

foot diameter excavated hole was allowed to remain open all the way to bedrock over the weekend prior to installation of the well casing and filter backfill. During that time the borehole was stabilized only by polymer slurry. Some time over the weekend, the dense silt caved-in to a level 23 feet above the bottom of the shaft.

Subsequent soil borings and pressuremeter testing indicated a significant loss of density in the silt around Well No. 3. Since it was felt that this loss of density could affect the adjacent caissons, a compaction grouting program was undertaken to re-densify the silt. Approximately 18 cubic yards of grout was pumped into the zone from the bedrock surface to approximately -90 feet CCD to more than equal the volume of the silt cave-in. Figure 2 shows a sketch of the planned well, the level of adjacent caissons, and the likely cave-in zone. Borings and pressuremeter testing were performed to evaluate the soil conditions following construction of Well No. 3 and the effectiveness of the compaction grouting. The test data is summarized on Fig. 3.

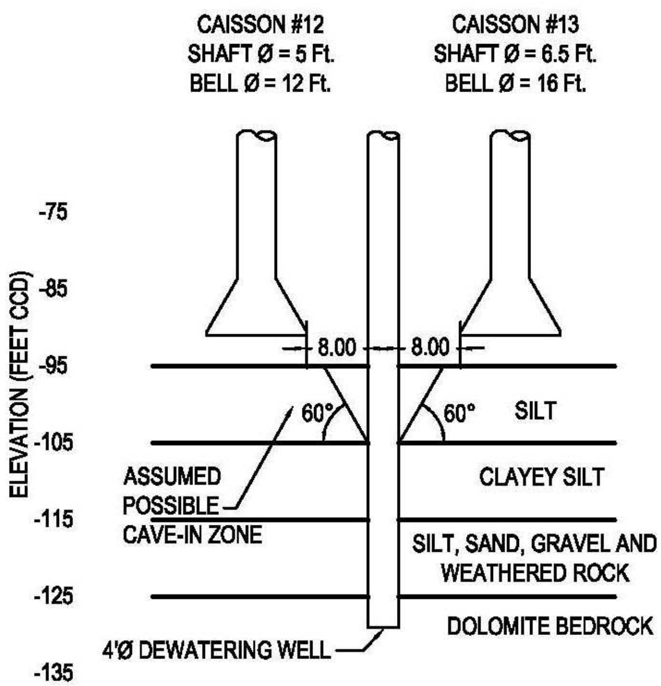


Fig. 2. Dewatering Well No. 3 Cross-Section

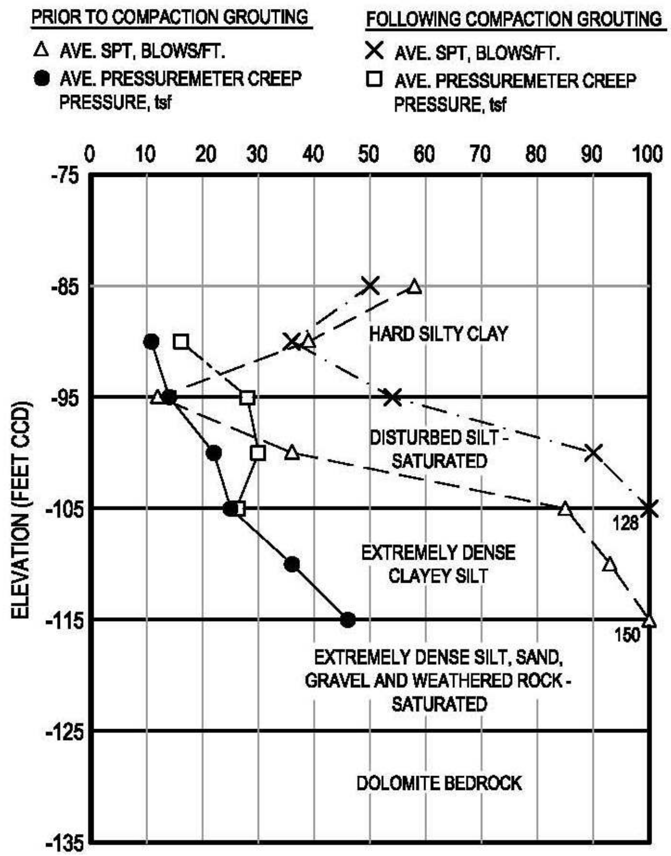


Fig. 3. Soil Profile Prior to and after Compaction Grouting

The locations of the caissons, wells and soil borings are shown on Fig. 4. The compaction grout injection points and grout take summary are shown on Fig. 5. The grout take volumes indicate that the cave-in was not necessarily uniform around the shaft.

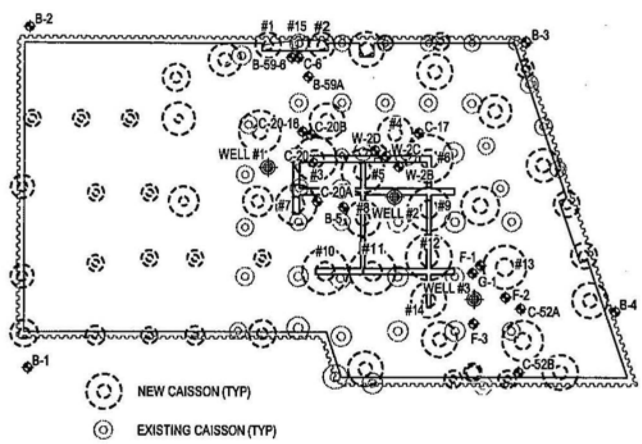


Fig. 4. Site Plan and Soil Boring Locations

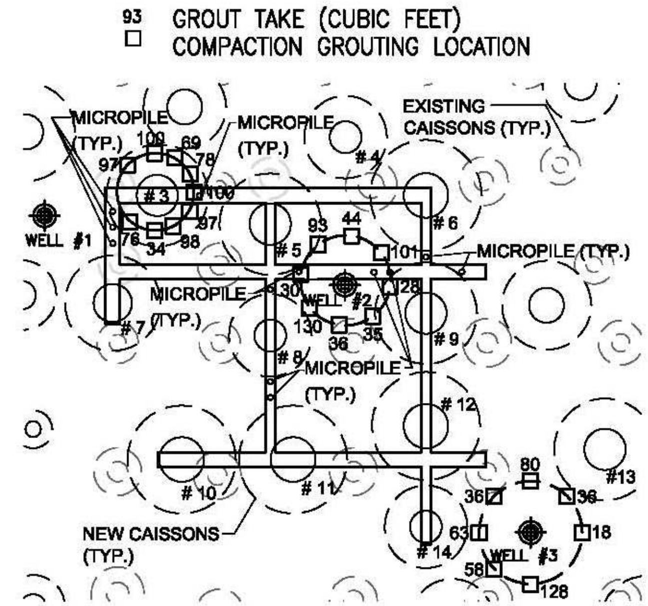


Fig. 5. Compaction Grout Point Locations

Well Nos. 1 and 2 were completed without incident. The wells consisted of one foot of sand filter around a 2 foot perforated steel casing with fine screening sized to keep the filter from entering the casing. Initial pumping from the dewatering wells was dirty, as expected but cleared up after several days. This indicated that the designed sand filter was working satisfactorily. The water head in this layer prior to pumping was approximately -13 feet CCD. However, since the total volume of water pumping was only in the range of 10 to 50 gallons per minute (gpm), the water head could only be lowered approximately 10 to 25 feet rather than the 40 to 50 feet which would be required to eliminate the possibility of a blow-in when constructing the caisson bells a couple of feet above the dense silt layer level. Unfortunately, the three properly filtered wells that extended into the dolomite apparently did not encounter sufficient cracks and fissures in the bedrock to permit effective lowering of the water table. The problem with dewatering wells for lowering the water head at a particular site in Chicago is that the bedrock is unpredictable enough that it is difficult to predict the number of wells required on the site. The number of wells to successfully lower the water table has varied from a minimum of one at a site where there was a very permeable gravel layer above the rock, to as many as six wells being required on another project.

Since the bells were being stopped just above dense silt in the thin hard clay layer, some caissons were being installed successfully at the design elevation for 48 ksf bearing (-92 feet CCD ±). However, others were not successful. Water problems were encountered deep within Caissons 3 and 15 which indicated that a change in design bearing elevation was necessary. Discussion was held on whether it made sense to add more wells to achieve the required drawdown based on the measured rates at the installed wells. However, the time involved, the relative lack of certainty on the number of wells required, and their effectiveness ruled this option out.

Problems at Caisson 3

Caisson 3 encountered water inflow and caving conditions when attempting to bell at the design elevation of -92 feet CCD. The water came in too fast for bailing so the partially excavated bell was plugged with grout extending up to the surface of the hardpan at which level another attempt to bell was made after allowing the grout to set because of the presence of existing old caissons and continued water seepage. It was not possible to adequately oversize the bell for the 36 ksf design bearing level. The as-constructed caisson consisted of an 11.5 foot bell at -71 feet CCD combined with the original 12 foot diameter bell plug at elevation -92 feet CCD as shown in Fig. 6.

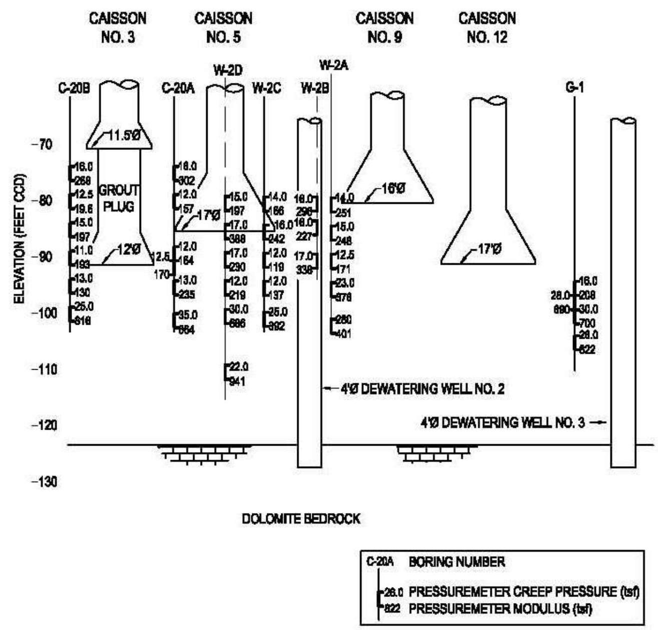


Fig. 6. Partial Cross-Section through the Core

Compaction Grouting at Caisson 3

In order to get the maximum capacity out of the as-built geometry, compaction grouting was undertaken beneath and adjacent to Caisson 3. The total volume of grout (38 cu.yd.) injected from near the bedrock to an elevation of approximately -90 feet CCD exceeded the estimated volume of loosened material that came into the excavation during the belling operation at elevation -92 feet CCD.

Pressuremeter Testing

Pressuremeter testing was performed around Well No. 2, Caisson 3 and Caisson 15 (which was actually replaced by Caissons 1 and 2). Because of the water problems encountered, there was some concern for loosening of the supporting silt. A check boring and pressuremeter testing indicated the marginal conditions at Caissons 1 and 2. Four compaction grout holes were recommended and installed to provide some insurance. The pressuremeter testing results near Caisson 3 and Well No. 2 are shown on Fig. 6.

Change in Design Required

The design for the remaining caissons was changed from 48 ksf at elevation -92 feet CCD to 36 ksf in the elevation range of -80 to -85 feet CCD. This decrease in allowable bearing pressure meant a number of the bells would need to be enlarged. Unfortunately, due to a communication mix-up, the required changes did not reach the operations personnel in the field in time and a number of the bells were put in undersized. Careful examination of the design loads allowed most of the undersized caissons to be considered acceptable even at the lower design bearing pressure. However, this mix-up resulted in a significant overstress at Caisson 9 and a slight overstress at Caisson 14. The overstress at Caisson 9 was sufficient enough to require supplementary micropile support with the micropiles serving as settlement reducers.

Dirty Water Pumping at Well No. 2 Next to Caisson 9

At the time that Caisson 9 was installed (at the higher bearing elevation), Well No. 2 suddenly started pumping dirty again. When the water did not clear after several days of pumping there was concern for ground loss and ground loosening from silt migration, so pumping from Well No. 2 was terminated. The calculated amount of material that could have been removed from the area during the dirty pumping was sufficient to require compaction grouting around Well No. 2 as a means of protecting adjacent Caissons Nos. 5, 6, 8 and 9. The amount of grout pumped approximately equaled the amount of calculated silt lost from the extended dirty water pumping.

Capacity Analysis and Settlement Predictions

The structural engineer and the geotechnical engineer worked together to determine the amount of added micropile support that was required to achieve desired performance levels for the proposed high-rise tower. The predicted settlements for the as-constructed caissons within the core of the building are shown in Table 1. The analyses were performed based on the testing information obtained following the compaction grouting program. Several alternatives and analyses were performed: shown are some cases with no additional micropile support and with 9, 12 and 17 additional micropiles. The maximum number of piles that could physically fit underneath the layout of grade beams as well as fit between the existing caissons was determined to be 17. The case with 12 additional micropiles, was the selected option.

The tall walls in the core area also act as grade beams, and with no openings in the first 22 feet, the core is extremely stiff. Consequently, the caissons in the core should settle equally. The settlements indicated in the table are based on theoretical loads and stresses without considering stiffness and stress overlap. The calculated individual caisson settlements in the core range from 1 inch to 1.7 inches. When allowance is made for the core stiffness and the apparent presence of a thin soft layer in the elevation range of -90 to -95 feet CCD, the core settlement without micropiles is approximately 1.5 to 2 inches. The settlement reduction provided by the micropiles ranged from 0.10 (9 micropiles) to 0.2 inches (12 and 17 micropiles).

PRESSUREMETER SETTLEMENT CALCULATIONS

DESCRIPTION			Dia. (ft)	p_s (tsf)	E_1 (tsf)	E_2 (tsf)	$E_{3/4/5}$ (tsf)	α_a	α_b	E_b (tsf)	w (in)
	Dead Load	Live Load									
Caisson #3	2337	380.5	9.1	19.4	177	640	797	0.50	0.50	370	1.3
Caisson #3, 4 micropiles			12.0	13.8	177	640	797	0.50	0.50	370	1.1
Caisson #5	5639	918	17.5	12.7	217	406	731	0.50	0.50	360	1.2
Caisson #6	5790	943	17.5	13.0	239	571	905	0.50	0.50	440	1.1
Caisson #7	5749	936	15.0	17.6	239	571	905	0.50	0.50	440	1.3
Caisson #8	4747	773	13.0	19.3	177	640	797	0.50	0.50	370	1.7
Caisson #8, 5 micropiles			13.0	11.1	177	640	797	0.50	0.50	370	1.0
Caisson #8, 4 micropiles			13.0	12.8	177	640	797	0.50	0.50	370	1.1
Caisson #9	6988	1138	16.0	18.8	227	207	551	0.50	0.50	270	1.9
Caisson #9, 4 micropiles			16.0	13.1	227	207	551	0.50	0.50	270	1.3
Caisson #10	4874	793	17.0	11.6	239	571	905	0.50	0.50	440	1.0
Caisson #11	5299	863	17.0	12.6	239	571	905	0.50	0.50	440	1.0
Caisson #12	5720	931	17.0	13.6	239	571	905	0.50	0.50	440	1.1
Caisson #14	2974	484	13.0	12.1	177	640	797	0.50	0.50	370	1.0
Caisson #4	4471	732	13.0	18.2	262	350	586	0.50	0.50	360	1.3
Group/no piles/no soft zone			65.0	8.6	540	10000	10000	0.50	0.50	1500	0.9
Group/9 piles/no soft zone			65.0	7.7	540	10000	10000	0.50	0.50	1500	0.8
Group/17 piles/no soft zone			65.0	6.9	540	10000	10000	0.50	0.50	1500	0.7
Group/12 piles/no soft zone			65.0	7.4	540	10000	10000	0.50	0.50	1500	0.7
Additional soft zone settlement											0.7

Notes:

$$w = \frac{1.33}{3E_b} p R_0 \left(\lambda_2 \frac{R}{R_0}\right)^{\alpha_b} + \frac{\alpha_a}{4.5E_a} p \lambda_3 R$$, where:

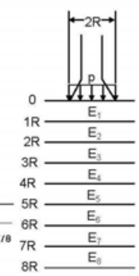
$R_0 = 1$ ft

$E_a = E_1$

$\alpha_{a,b} = E_d/E^+$

$\lambda_2 = \lambda_3$ = Shape Coefficients = 1.0 for Round Foundations

$$E_b = \frac{3.2}{\frac{1}{E_1} + \frac{1}{0.85E_2} + \frac{1}{E_{3/4/5}}} \quad \text{or} \quad E_b = \frac{3.6}{\frac{1}{E_1} + \frac{1}{0.85E_2} + \frac{1}{E_{3/4/5}} + \frac{1}{2.5E_{6/7/8}}}$$



Considering individual caissons, without adjacent load transfer benefit, the effect of piles concentrated at a caisson location has a more significant effect. For example, at Caisson 3 the four supplemental piles reduce the individual calculated settlement from 1.3 to 1.1 inch. At Caisson 9, four supplemental piles reduce the predicted settlement from 1.9 to 1.3 inches.

The pressuremeter testing was typically performed a minimum of 48 hours after completion of compaction grouting at a specific location. This is probably sufficient for fine sands and silts, but it may not be sufficient for clayey silts and silty clays. Where the soil has some clay content the compaction grouting undoubtedly builds up excess pore pressures which have insufficient time to drain in two days and thus a lower test result is obtained than would be obtained if the test were run several weeks later. Thus, wherever a test was performed in clayey silt the actual pressuremeter values will undoubtedly be much higher than tested. Both of these factors should mean that settlement predictions based on the current test data should be conservative and the actual settlements should be less than predicted. Additionally, no skin friction has been taken into account in the settlement predictions, as is typical practice in downtown Chicago. However, testing in the area has shown significant loads carried by skin friction.

Settlement Observations

At the time of the writing of this paper, the tower is almost topped out with 90 percent of the dead load in place and settlement observations are significantly less than predicted. This indicates that some ground stiffening occurred after compaction grouting and the in-situ pressuremeter testing which formed the basis of the revised settlement predictions.

Lessons Learned

1. Clear communication channels between the design engineers and the field are an absolute necessity particularly when changes are necessary during construction.
2. Assuming normal polymer slurry will maintain 4-foot diameter shafts open for 145 feet (with the bottom 30 feet extending through dense silt) is risky.
3. Compaction grouting can be an effective method for redensifying silt loosened by shaft cave-in.
4. Things are not always what they seem. Conditions can vary drastically over short distances. Successful completion of small caissons at a site many years earlier is no assurance for the same construction of new large diameter bells many years later. Unpredictable meandering sand channels at hardpan interface could have been connected to the deep rock water table through original borings inadequately grouted or by unknown hand dug ringed and lagged caissons.
5. Appropriately placed supplemental micropile support to bedrock can provide predictable settlement reduction because of the known modulus of steel.
6. The fact that observed settlements were significantly less than predicted settlement could mean that the compaction grouting effect was tested too soon and that with pore pressure dissipation the ground became stiffer.
7. Designing the number of wells and the filter system to inhibit silt migration and carry sufficient water to adequately lower the deep rock water table is very difficult, and it requires extensive analysis and site-specific testing.
8. Whenever existing caissons are present, there is a possibility of actual locations varying from locations shown on the existing building drawings. Strategies to handle this possibility should be determined and budgeted for ahead of construction.

CONCLUSION

Unanticipated deep foundation construction problems can be successfully overcome with cooperation between the owners, engineers and contractors. Appropriate in-situ testing can help to determine the new foundation properties, and when combined with ground modification, the need for supplemental support can be minimized to ensure a positive conclusion to the project.

REFERENCES

Baker, C.N., Jr. [1993]. “Use of Pressuremeter in Mixed High Rise Foundation Design”, ASCE Special Bulletin No. 38.