PARKING GARAGE SUPPORTED ON RE-USED CAISSONS AND COMPOSITE CAISSON-MICROPILES

Seth Robertson, Ph.D., P.E., GRL Engineers, Inc., 2 Washington Square, P.O. Box 1285, Haverhill, Massachusetts, SRobertson@GRLengineers.com
James Davis, P.E., GZA GeoEnvironmental, Inc., 249 Vanderbilt Avenue, Norwood, Massachusetts, James.Davis@GZA.com
Lawrence Johnsen, P.E., S.E., GZA GeoEnvironmental, Inc., 249 Vanderbilt Avenue, Norwood, Massachusetts, Lawrence.Johnsen@GZA.com
Alexander Ciccone, P.E., Keller – North America, 101 Centerpoint Drive Suite 219, Middletown, Connecticut, Alex.Ciccone@Keller-NA.com

ABSTRACT

The most sustainable foundation is the one that is already in place. That doesn't mean it is the easiest to RE-use. This was realized when replacing a multi-story hospital parking garage in Hartford, Connecticut. The existing garage was constructed in the 1980s and was supported on grade beams with 3- and 4.5-foot-diameter drilled shafts that extended through 100+ feet of problematic soft Connecticut Valley varved clay, glacial till, and end-bearing in or on arkose bedrock. The proposed replacement garage had relatively high loading demands, was susceptible to excessive settlements, and was also located adjacent to an active hospital with sensitivity to vibrations, thereby limiting many foundation design options. An innovative solution was proposed to re-use many of the existing caissons, supplemented with a combination of micropiles and composite foundations. Composite foundations were comprised of an upper 3-foot-diameter drilled shaft with a deeper central, 11.75-inch-diameter micropile to transfer structural axial loads to bedrock. Installation information for the existing caissons was heavily limited. Dynamic load tests were therefore performed on 12 existing shafts spread across the garage footprint, confirming geotechnical resistances up to 4,500 kips and significantly improving quality assurance of the existing foundation construction. Additionally, two micropile load tests were performed in tension to 900 kips, confirming design loads could be resisted by the bond strength in rock. The foundation re-use methodology saved over 3 million dollars in installation costs, weeks of construction schedule, and resulted in a significant carbon reduction compared to conventional, and inherently more substantial, foundation replacement strategies.

BACKGROUND

St. Francis Hospital has served the Hartford Connecticut area since 1885. Through the years it has expanded to become one of the largest hospitals in New England. New construction for the hospital facility over the past 50 years has been founded on mat foundations, H-piles, and caissons. Due to the local significance of the hospital facility, further improvements were necessitated but were limited in available space and potential impact on the existing active structures. The complete replacement of a deficient parking garage was proposed, which required innovative construction practices to overcome several limitations.

The new garage replaces a four-level parking garage located along Collins Street originally constructed in 1983. The footprint of the new garage is identical to the old garage. The old garage was supported on 40 3-foot-diameter, and 26 4.5-foot-diameter caissons that extended up to 3.5 feet into bedrock. The design drawings for the old garage specified service axial capacities that varied from 1,075 to 1,450 kips for the

3-foot-diameter caissons and from 2,150 to 2,700 kips for the 4.5-foot-diameter caissons. However, the project Structural Engineer (Walker Consultants) estimated that the actual axial loads varied from 480 to 1,249 kips for the 3-foot-diameter caissons, and from 1,200 to 2,158 for the 4.5-foot-diameter caissons.

Subsurface conditions generally consist of 5 to 10 feet of miscellaneous fill, varved clay that extends to depths of up to 150 feet, 10 to 50 feet of glacial till, and bedrock. The clay has a thin, hard crust overlying compressible clay. The successful mat foundations on the hospital campus have limited the increase in overburden pressure to 200 psf for settlement control. Bedrock is mapped as Portland Arkose which consists of arkose sandstone or shale. The arkose sandstone contains fragments of igneous rock which can be problematic during drilling operations. The complications associated with the complex subsurface conditions and proximity to the active hospital therefore compelled unique foundation design and construction solutions.

PROPOSED PARKING GARAGE FOUNDATION DESIGN

Foundation Design Considerations

The proposed new foundation for the new garage is supported on 25 of the existing 3-foot-diameter caissons and 21 of the existing 4.5-foot-diameter caissons, as well as 13 new micropiles and 12 new composite foundations. The composite piles consisted of a 3-foot diameter, 20-foot-long reinforced drilled shaft with a central 11.875-inch diameter micropile. The micropiles, both as part of the composite piles and the discrete micropiles, extended into bedrock. Maximum axial service loads were 916 kips for the existing 3-foot-diameter caissons, 1,688 kips for the existing 4.5-foot-diameter caissons, 527 kips for the new composite piles, and 523 kips for the new discrete micropiles. Maximum service lateral loads were 89 kips for the existing 4.5-foot-diameter caissons, 91 kips for the new composite piles, and up to 18 kips for the new discrete micropiles. The existing 3-foot-diameter caissons are located primarily under the exterior walls and do not receive lateral load.

The reuse of caissons was critical to the project budget. The Design Team originally was basing the design of the new garage on supplementing existing caissons with full-depth new caissons socketed into bedrock, which were budgeted at $1,200 per foot for lengths of 120 to 150 feet. Even with the reuse of 46 existing caissons, the project was over budget. At that point, value engineering was required for the project to move forward.

Micropiles could support the axial loads at approximately 30% of the cost of new full-depth caissons, but were not capable of resisting the required lateral loads. However, with a 3-foot-diameter shaft for the upper 20 feet, the composite pile concept was able to accommodate the axial loads while resisting the significant lateral loads. The composite caisson-micropile was introduced and added value to the project through cost savings (approximately on the order of $1.5 million) and schedule savings (estimated at least 4 weeks).

Additional benefits of the caisson re-use and the introduction of the composite piles were savings in the carbon generated from new foundations. When reviewing a side-by-side carbon calculation, it was demonstrated that the lowest-cost option would also generate one-third of the carbon compared to complete foundation replacement with drilled shafts.

New Foundation Design Demands and Methodology

The unique design challenges included: (1) Selection of existing caissons to load test, (2) load transfer between the caisson and micropile in new composite piles, (3) axial and lateral strain compatibility between new and old foundations of varying size and load, and (4) Building Code waivers for Grade 80 rebar, modified load testing of new foundations, and verification testing of existing caissons.

Available information on the existing caissons was limited to the original design drawings which provided proposed depths, loads, and steel reinforcing details. Installation logs and any load testing results were not available from original construction. The International Building Code ("IBC") 2021 Section 1810.1.2 (Connecticut State Building Code) does not allow reuse of existing foundations unless satisfactory evidence is submitted to the Building Official, which indicates that the elements are sound and meet the requirements of the IBC. A total of 46 caissons were planned for reuse. GZA proposed that 25% of the caissons be load tested to verify ultimate capacities of at least twice the proposed service load. A waiver was submitted and approved by the Building Official describing the proposed reuse and testing of existing caissons. The caissons selected for load testing were based on comparing estimated current loads to new proposed loads, distributing testing to both 3- and 4.5-foot diameter caissons, and distributing the testing across the garage footprint. In general, the new loads on the existing caissons were less than 85% of the estimated loads from the existing garage.

Lateral load analyses on existing and new foundations were performed using LPile to determine whether existing caissons could resist the lateral loads from the new garage, the required flexural reinforcing and depth of the caisson element for composite piles, and potential issues associated with strain compatibility for the composite foundation sections. The maximum moment on any foundation element was 2,754 inch-kips occurring at the bottom of the grade beam. The tops of the caisson elements extended two feet into a grade beam to provide a fixed-head connection. The new garage was designed for east-west lateral loads to be resisted by grade beams and north-south lateral loads to be resisted by enlarged pile caps. The grade beams were supported by foundations of varying size ranging from about 12-inch diameter to 4.5-foot diameter. Since the grade beams restrain the foundation elements, allowing uniform deflection, the lateral load resisted by each foundation was estimated based on its lateral stiffness. Group effects were also considered in the load distribution and deflection analysis for each pile due to the foundation sizes and spacing. It was determined the existing caissons were sufficiently reinforced to resist the lateral loads. A reinforcing cage was required for the upper 20 feet of each composite pile. Below 20-foot depth, the maximum moment was less than 60 inch-kips, which was resisted by the micropile portion of the composite pile.

Axial loads on composite piles varied up to 536 kips compression and 115 kips tension. The discrete micropile axial load varied up to 536 kips compression with no tension demand. All micropiles (discrete piles and the portion of the composite piles) consisted of a Grade 80, 11.75-inch-diameter (0.672-inch wall thickness) steel casing seated into competent bedrock. Rock sockets were 10-inch diameter and up to 25 feet long for supporting the heaviest loads. Rock sockets were reinforced with either five #14 grade 80 rebar, three #14 grade 80 rebar, or a single #24 grade 80 rebar. Piles were filled with 5,000 psi grout.

The Connecticut Building Code (IBC) limited usable yield stress for micropile reinforcement in compression to 75 ksi. A waiver was submitted and approved by the Building Official, noting that the current American Concrete Institute ("ACI") 318-19 permits 100-ksi usable yield stress in compression, when special confinement is provided, and 80 ksi when not provided. The Project team was approved to use 80-ksi rebar with an allowable compressive stress of 32 ksi.

Bedrock consists of arkose sandstone and shale. The initial test borings did not extend more than five feet into bedrock because it was anticipated that caisson foundations would be used for the new parking garage. The bid documents assumed a conservative allowable bond stress of 50 psi resulting in a 41-foot-long bond zone for the larger axial loads. The bid documents required the contractor to perform additional test borings deeper into bedrock and allowed adjustments to the bond length based on the results of instrumented load tests.

Static Axial Load Test Results

Performing a static axial compressive load test to double the design load of over 1,000 kips would have required a significant reaction load frame, which would be very expensive, time-consuming to install considering the depth of reaction anchors. Tension load tests were specified as they can be performed at lower costs with a smaller load frame, and in some cases without the need for installing reaction piles. Figure 1 presents a photograph of the tension load test setup. The limitation of the tension load test is the maximum yield strength of a threaded bar (Grade 80, #32 bar) is only 1,004 kips, which was less than 2 times the required design load. The load test program included two tension tests (ASTM D3689 - Quick Test) on 20-foot bond lengths to verify ultimate bond stresses. Each test was taken to a maximum load of 900 kips, 90% of yield stress for a Grade 80 #32 threaded bar. The test micropiles were specified to have a smaller bond length to confirm the original design bedrock bond strength (50 psi), but to also test higher bond strengths which could be used to reduce production rock sockets. The approach of testing the bedrock bond strength on shorter socket lengths and smaller test loads than two times the design load required a design waiver which the Building Official approved.

Fig. 1. Micropile Tension Test Setup (Test #1).

The test micropiles were the same size as the production micropiles, except a full length #32 center bar was installed to accommodate the maximum designated test loads. One of the challenges of tension load testing was locating available jacks, load cells, and accessories that could accommodate a #32 bar (4-inch diameter). Fortunately, the available equipment conforming to ASTM standards was eventually located, with the exception to a hemispherical bearing plate, which was allowed by the Design Team not to be used. At a maximum test load of 900 kips, the two test piles deflected between 1.7 and 1.8 inches, in which about 1.3 to 1.4 inches was elastic deflection of the steel. The load testing confirmed an allowable bond strength of the bedrock of 65 psi, which was higher than the original bidding bond strength of 50 psi and production pile socket lengths were reduced by about 25%.

Each test micropile included sisterbar strain gauges to evaluate load distribution characteristics in the clay, glacial till, and bedrock. Strain gauges were installed in pairs at approximate depths of 5, 48, 116, 139, 143, and 152 feet below grade. The soil profile generally consisted of fill to 15 feet, varved clay to 115 feet, glacial till to 126 feet, fractured rock to 138 feet and competent rock below 126 feet. The project’s specifications required several levels of strain gauges in the rock socket due to the length of the pile and concern for gauge survivability during installation. The pile casing was extended through the fractured rock. Since the test was run in tension, the effects of grout cracking on the pile elastic modulus were reasonably approximated in computing pile internal forces. The computed internal forces determined from strain measurements for one of the test piles is shown in Fig. 2.

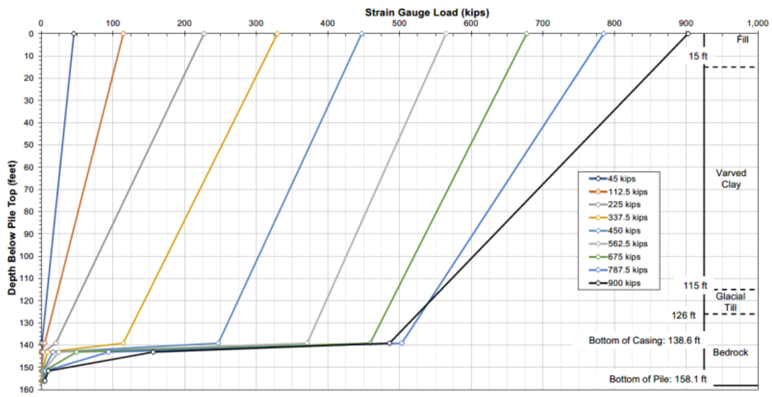

Fig. 2. Internal Force Versus Depth Determined from a Tensile Static Load Test.

For each test pile, at least one set of strain gauges in the rock socket was non-functional after the installation of the rebar. The strain gauges were bench-checked prior to installation and during the process of installation, however damage still occurred. The rebar was installed in 20-foot segments due to the weight of the #32 bar and the cables needed to be kept clear of the rebar during splicing, but then secured to the rebar after splicing. The process of splicing and potential for kinking/cutting the cables on the outer casing was assumed to be the cause of damage.

Measured strains in the Glacial Till and Varved Clay strata were neglected from the computed internal force profiles to provide an estimated average resistance over the cased zone. The geotechnical resistances within these strata were not directly accounted for in the foundation design, however strain gauge instrumentation was implemented for potential redundancy and prudence in assessing the load-distribution characteristics along the full pile lengths. Overall, an activated bond stress of 130 psi was conservatively estimated in the competent rock which resulted in the allowable value of 65 psi used for design.

LOAD TESTING OF EXISTING CAISSONS

Test Selection and Methodology

Available options for testing the twelve existing caissons were limited to static load testing per ASTM D1143, Statnamic testing per ASTM D7383, and High-Strain dynamic load testing (“HSDLT”) per ASTM D4945. The cost of even a single static load test on an existing caisson in compression to a test load of 3,400 kips would have been prohibitive and virtually impossible to achieve due to the confined site. The relative costs of HSDLT compared to Statnamic testing were not directly evaluated due to the lack of availability of Statnamic systems. Therefore, HSDLT was chosen on the basis of availability, schedule, and cost compared to static load testing alternatives. The cost of HSDLT on twelve caissons was under $140,000 (including engineering services and equipment shipping/transportation costs), which equates to under $12,000 per test caisson.A free-fall, drop-weight hammer (i.e., APPLE) was utilized for accommodating controlled impacts while carefully monitoring impact stresses and indications of the total mobilized soil resistances. HSDLT generally entails measurement of the deep foundation response to an

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impact load on the foundation head. HSDLT was conducted in accordance with ASTM D4945 with dynamic measurements obtained using a Pile Driving Analyzer ("PDA"). Dynamic measurements of strain were taken during each impact using eight strain transducers on a top force transducer (i.e., load cell), averaged in four orthogonally spaced pairs inside and outside of the central steel cylinder. The load cell was anchored onto the impact surface to limit movement during the testing sequences. Measurements of acceleration were collected using at least two accelerometers attached to the above-grade foundation portion approximately 32 inches below the load cell. Where possible, additional accelerometers and/or strain gages were instrumented on the shaft head below the cap. Strain and acceleration signals were conditioned and converted to forces and velocities by the PDA. The original caisson installation records were not available. Shaft lengths were therefore estimated based on the available subsurface explorations, assuming at least one-diameter socket length in competent rock.

Several benefits of HSDLT over alternative load testing methods were considered for quality assurance program development. Primarily, HSDLT permitted testing multiple foundations across the site over a relatively short period of time. This increased confidence in the quality of the as-built caissons without any detailed installation records from the original construction. Additionally, resistance distribution characteristics were assessed based on the dynamic measurements without the need for embedded instrumentation.

Test Program Summary

Twelve existing caissons were selected for dynamic testing across the project location. The tested caissons included six 3-foot-diameter shafts along the exterior perimeter of the site, and six interior 4.5-foot-diameter shafts. Designated test loads were 1,832 kips and 3,376 kips for the 3- and 4.5-foot-diameter caissons, respectively. The test loads were selected based on twice the maximum designated design loads.

Preliminary dynamic testing simulations were conducted with a wave equation analysis program to estimate the required ram weight, drop height, and plywood cushioning required to mobilize the designated test loads while minimizing impact stresses. For versatility in testing the various caisson sizes and designated test loads, a drop-weight system was selected with a modular 32-ton ram weight and drop height up to three feet.

Special considerations were required for the proper implementation of HSDLT on existing caissons. Dynamic testing requires a smooth, flat impact surface, and ideally no major geometric and material changes near the impact location. Excessive effort was realized to expose the caisson head, requiring additional excavation which would potentially expose weak clayey strata and impose extensive soil stabilization and dewatering efforts over the testing duration. The foundation caps were therefore exposed and modified to optimize the dynamic measurements and associated analyses/interpretations. Modifications included removing the existing grade beams, grinding down any large concrete protrusions, and grouting a smooth surface on the top of the exposed caps. This approach protected the existing steel reinforcement embedded in the foundation cap to tie into the future structure.

Complications were incurred for interpretation of the dynamic measurements, as well as drop weight system setup due to the geometric changes near the instrumentation locations. The quality and size of the foundation caps varied across site. A 4-foot by 4-foot square cap was exposed for most 3-foot-diameter caissons. Four 4.5-foot-diameter caissons included a 5.5-foot-diameter circular cap, while the other two caissons were exposed with a 5.5-foot wide square cap. The enlarged square cap was not anticipated prior to equipment mobilization, primarily since excavations were being performed concurrent with the dynamic testing, which required modifications to the drop weight system. The cap was completely removed for one 3-foot-diameter caisson (TS12-A3.2) which accommodated dynamic measurements directly on the caisson head. Figure 3 and Fig. 4 present several example images for HSDLT conducted on the 3- and 4.5-foot-diameter caissons, respectively.

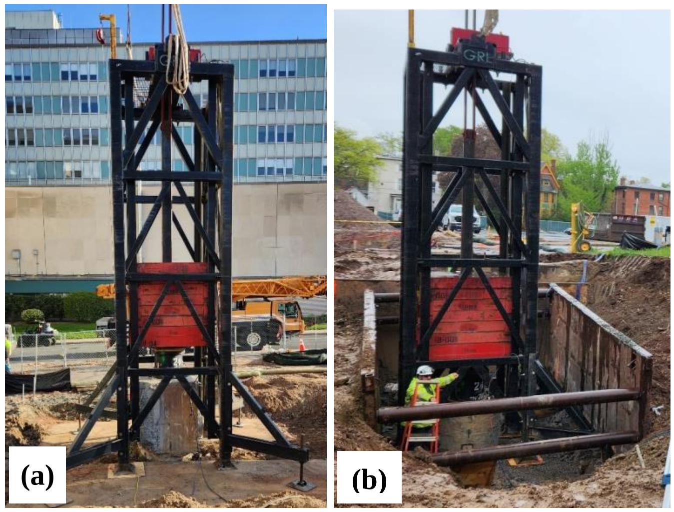

Fig. 3. Imagery of HSDLT on 3-foot-diameter caissons: (a) TS3-D1.0. (b) TS12-A3.2.

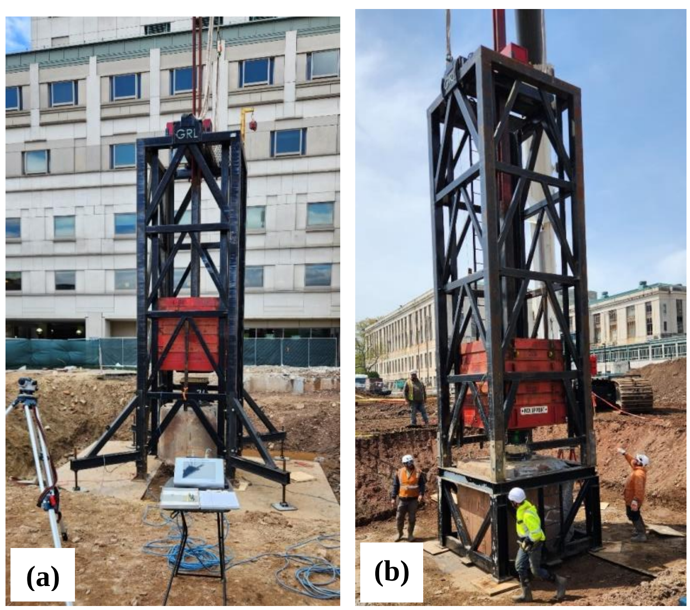

Fig. 4. Imagery of HSDLT on 4.5-foot-diameter caissons: (a) TS1-B1.0. (b) TS11-C8.1.

Test Results

For each caisson, the HSDLT was conducted under several impacts with progressively increasing drop height from 0.5 feet to a maximum 3.0 feet. Overall drop height was limited where the foundation caps extended inside the APPLE guide frame, but this generally did not have any negative impact on the overall testing scope. The permanent caisson displacement per impact was monitored using a site level and generally remained under 1/32 inch. Plywood cushioning was placed on top of the load cell and was adjusted throughout the testing sequence based on impact stresses monitored on the PDA. Test caissons were numerically designated in the order in which they were tested, from TS1 through TS12, followed by the original structure column layout. For example, the first caisson tested was TS1-B1.0 and was located at column line B1.0. Table 1 summarizes the pertinent test caisson details and HSDLT results.

The mobilized soil/rock resistances were monitored during testing based on the total calculated impact forces, maximum displacements, and transferred energies. The force and velocity signals under selected impacts were then analyzed in the CAPWAP signal matching software for computing the total mobilized resistances and resistance distribution along the embedded caisson lengths. The calculated mobilized resistance exceeded the designated test loads on the analyzed impact for all tested caissons. Given the minimal permanent set observed during the testing sequences, these results likely reflect minimum ultimate capacities. Maximum stresses over the total caisson lengths remained below typical limits.

Table 1. Summary of Pertinent Test Caisson Details and HSDLT Results

Caisson No.	Nominal Caisson Diameter (ft)	Approx. Cap/Head Dimensions (ft)	Drop Height (ft)	CAPWAP Mobilized Resistance			Maximum Stresses	
				Total (kips)	Shaft (kips)	Base (kips)	Comp. (ksi)	Tens. (ksi)
TS1-B1.0	4.5	ø5.5	2.2	3,450	3,005	446	1.6	0.4
TS2-C1.0	4.5	ø5.5	2.4	3,450	2,948	502	2.0	0.3
TS3-D1.0	3.0	□4.0	1.5	2,660	2,448	212	2.0	0.4
TS4-C3.1	4.5	ø5.5	2.0	3,600	2,011	1,589	1.7	0.2
TS5-B3.1	4.5	ø5.5	2.1	4,000	3,627	373	1.5	0.3
TS6-D6.1	3.0	□4.0	1.5	2,450	1,483	967	1.7	0.6
TS7-D9.1	3.0	□4.0	1.5	2,365	1,798	567	1.2	0.3
TS8-A9.1	3.0	□4.0	1.5	2,360	1,967	393	1.9	0.3
TS9-A6.1	3.0	□4.0	1.5	2,150	1,941	210	1.1	0.3
TS10-B8.1	4.5	□5.5	3.0	4,200	3,489	711	1.5	0.2
TS11-C8.1	4.5	□5.5	3.0	4,500	3,951	549	1.5	0.3
TS12-A3.2	3.0	ø4.5	1.5	2,000	1,699	301	1.3	0.3

Special considerations were required for the signal matching analyses. The below-grade caisson was exposed where possible to obtain approximate measurements of the reduced cross-section at the caisson head. Non-uniform impedance variations were modelled in CAPWAP to simulate the known geometric changes, with assumed concrete overconsumption (approximately 120-140%) over the embedded shaft lengths. Due to the uncertainty in the in-place concrete strength and elastic properties, additional dynamic measurements were obtained on the caisson head just below grade for comparisons of the dynamic measurements with assumed stress wave speeds and concrete elastic moduli. For instance, strain and acceleration (in addition to force transducer measurements) was measured both on the foundation cap and the caisson head for TS7-D9.1. This allowed for comparing various force calculation methods, and assessment of the load transfer characteristics from the dynamic impact into the embedded caisson. Overall, the HSDLT program successfully confirmed the existing caissons could withstand the proposed service loads. Despite the uncertainties in the as-constructed caisson properties and additional HSDLT constraints, there were no major issues encountered in obtaining sufficient dynamic measurements on the existing caps and subsequent signal matching analyses for assessing the caissons' load bearing capacities.

INSTALLATION OF COMPOSITE PILES

The main challenge for the composite caisson-micropile solution, aside from complexity in design, was the sequence of installation. Several options were considered and reviewed for cost and feasibility. The selected process began with drilling in a temporary casing for the 3-foot-diameter caisson portion, and performing clean-out procedures after drilling to 30-foot depth. Due to concerns with the clean-out procedures in some instances, the temporary casing was alternatively over-drilled by several feet, backfilled with flowable fill or concrete to create a "plug", and then the casing was pulled up to top of flowable fill. Next, the top of the temporary casing was covered with a steel plate with a 11.75-inch diameter central hole. The micropile was then drilled using wet rotary duplex methods down through the plug and continuing to bedrock. Once the micropile rock socket was drilled and cleaned out, the reinforcement was inserted and the pile was tremie-grouted such that a clean, undiluted grout return was observed at the top of micropile. For this process, the hole was covered with plywood and sealed with foam to reduce the slurry and grout return from entering the caisson portion.

The caisson installation proceeded with the remaining spoils and grout above the plug removed from inside the temporary casing by means of vacuum-excavation. It should be noted that this method would have been unable to excavate spoils if the bottom of shaft was much deeper than 30 feet. Once the hole was cleaned, the cage was set to the bottom and concrete was poured while pulling the temporary casing. Figure 5 shows the cage in place prior to final concrete placement.

Fig. 5. Composite caisson-micropile prior to concrete placement.

Alternative installation methods were also considered but were not ultimately adopted due to potential risks associated with each. One option was the possibility of drilling and pouring the caisson portion first. For this sequence to be possible, a PVC or steel sleeve would be required to accommodate subsequent micropile drilling operations. However, this was abandoned due to the need to clean and grout the small annular space between the caisson concrete and micropile for effective lateral load transfer. When drilling the micropile, the annular space would potentially fill with clay spoils and removal of those spoils would prove difficult. Additionally, the possibility of movement of the sleeve when pouring the caisson would be detrimental to the pile installation process and would require consistent rework. Vibratory installation of the temporary casing around the installed micropile may have been feasible if combined with vacuum excavation, but this was not implemented due to the proximity to the active hospital building.

Due to the proximity of the active hospital, vibration monitoring was implemented on and nearby the adjacent hospital structures throughout the load testing durations and construction sequences. There was no indication from the vibration monitoring data nor visual inspection that indicated any detrimental effect on the existing structures throughout the project's duration. The selected installation sequence for the composite caisson-micropiles proved to be an effective and innovative solution given the project's site constraints and design demands. The installations remained on schedule throughout the project duration, reinforcing the ultimate savings in cost and schedule when coupled with the foundation re-use practices.

SUMMARY AND CONCLUSIONS

The St. Francis Hospital project involved demolition of an existing parking garage, and reconstruction of a new garage in its original place. Due to site space limitations, complicated subsurface stratigraphy, overall budget constraints, and locality to the active hospital, innovative foundation design and construction practices were utilized. The original parking garage was primarily supported on caissons extending, or socketed, into competent arkose sandstone bedrock. Available caisson installation records were minimal, however the caissons were estimated to support higher service loads than proposed for the new structure. Many of the existing caissons were therefore proposed to be re-used in lieu of constructing new foundations. High-strain dynamic load tests were conducted on 12 of the existing caissons to designated test loads at least twice the anticipated new service loads. New deep foundations were still required, however constructing rock-socketed caissons was cost prohibitive and complex with the site limitations. Alternative composite caisson-micropiles were designed to withstand large lateral load demands while sufficiently transferring axial loads to the bedrock formation. Static axial tensile load tests were performed to confirm micropile bond strengths and to optimize the required bond lengths in competent rock. Based on the unique design strategy and load test program, several conclusions can be made:

1. Significant cost and schedule savings resulted from reusing 46 existing caissons and constructing new composite caisson-micropiles. The cost of a new foundation, supported entirely on new caissons, would have been more than double the cost and schedule duration, and may have resulted in the project termination. Overall, at least 3 million dollars in installation costs and weeks of construction schedule were saved compared to complete replacement of the existing foundations.
2. High-strain dynamic load tests were conducted using a 32-ton drop hammer, and upward of three-foot stroke. The dynamic testing did not cause any damage to mat-supported structures located as close as 50 feet to the testing. Axial compressive resistances of 2,000 to 4,500 kips were verified on the existing caissons.
3. Tension testing to failure on reduced socket lengths allowed verification of a higher ultimate bond strength of bedrock without exceeding the tensile capacity of threaded bars.
4. The optimal installation of the composite caisson-micropiles involved installing the micropile portion in the middle of the excavated caisson prior to concrete placement. Alternatives were assessed and deemed impractical due to potential quality reductions incurred from drilling spoils between the various caisson/micropile installation methods.

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