30 Hudson Street
Foundation Design and Construction in Variable Rock

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A new office complex is being constructed within 25m of the Hudson River in New Jersey. The project, about 140m by 100m, will include 4 basements under the entire footprint. The perimeter was to be secured by a 0.8m thick diaphragm wall, laterally supported by high capacity tieback anchors to rock. Columns were to be supported on footings or piers to rock at a nominal bearing pressure of 3.8MPa at depths up to 17m. Although some variation in rock quality was observed in borings, a substantial vertical seam of weathered and/or decomposed rock, 1 to 2 meters wide by as much as 8 meters deep, was encountered during construction necessitating field redesign. In some locations, the seam dipped below rock that is more competent. Footings were redesigned for support on drilled caissons, or soft rock was excavated and replaced with concrete necessitating a reduction in bearing pressure of the footings by one half. The tower covering the south half of the site was topped out in 2002 and the garage levels in the north half of the site were completed in 2003.

PROJECT DESCRIPTION

In September 1999, a major investment company began the due diligence process to evaluate a property on the shore of the Hudson River in Jersey City, New Jersey for a new multi-use complex. The site offers a spectacular view of lower Manhattan directly across the river from the site of the former World Trade Center. The site, covering two city blocks, was previously occupied by manufacturing and office facilities of Colgate-Palmolive.

The building footprint is approximately 140m by 100m, the southern half of which is occupied by a 260m tall office tower, shown in Fig. 1. As of this writing, design of the structure for the northern half of the site has not been finalized, but it is expected to comprise an 11-story office building. A glass-covered atrium will occupy the street that formerly separated the two blocks. As part of the due diligence process and preliminary design evaluation, alternatives were considered to accommodate various numbers of parking places, both above and below existing grade. A major consideration for below grade parking is the proximity of the Hudson River some 25m east of the east building line.

SITE HISTORY

As late as 1804, the Hudson River shoreline was roughly parallel to the west side of Hudson Street one block west of the present shoreline. Land was created by the construction of timber crib bulkheads at about the existing shoreline. Cribs were filled with boulders and miscellaneous debris and the interior of the site was filled with ash, cinders and soil. Prior to its most current use, the site was occupied by various small industrial and commercial establishments. Colgate began operations in Jersey City in 1847 and expanded into the areas to be occupied by the new tower.
The southern portion designated Site 4, was most recently occupied by a large warehouse supported on a 180mm thick slab supported by 560mm concrete filled steel shell pipe piles with 1200mm thick pile caps. Top of slab was about 3.5m above mean sea level (MSL). The northern portion of Site 4 and most of Site 3 were occupied by pile supported tanks and other industrial facilities. The latter facilities were also supported by concrete filled steel pipe piles varying from 300 to 560mm in diameter, driven to rock. Top of slab for the buildings was between Elev. 2.75 and 3.5m above MSL. In the late 1990’s, a pile-supported esplanade and box culvert were constructed outside the timber crib bulkhead wall. Lateral support was provided by a pile supported deadman and steel tie-rods. River flood levels have reached 3.2m above MSL.

**SITE GEOLOGY**

The site lies just west of the Hudson River which divides the older Precambrian rocks to the east from the Triassic rocks to the west, except in the lower stretch of the river near Jersey City where Precambrian rock is found, probably extending less than a mile west of the river. Bedrock is part of the New York City group of the Manhattan prong. The group comprises a series of metamorphic rock types, which include Schist, Gneiss and Marble. The Schist rock type, known as the Manhattan Formation underlies the project area.

The ancient Hudson River is thought to have originated approximately 60 million years ago. The continental glaciation gouged the river channel down to the unweathered rock. During various glacial stages, outwash sands and/or glacial till were deposited. At the end of the Pleistocene as the sea level rose, a brackish environment developed in which the present river bottom organic silt and peat layers were deposited. Fills were added during the past two centuries.

Existing grade is about Elev. 7m at the west side of the site sloping downward to about Elev. 1.75m at the river edge. The 100-year flood level is Elev. 3.04m as determined by FEMA.

**SUBSURFACE INVESTIGATIONS**

Initial studies were based on available borings dating from as early as 1902 but as current as 1998. Hence, some of the data, particularly from the earliest borings were not considered completely reliable. Drilling and sampling techniques used were, in some cases, crude and undocumented.

The general subsurface profile as shown in Fig. 2, derived from the available borings was not significantly different from the subsurface profile developed from the preliminary and final subsurface investigations but it was the details derived from those investigations that differed significantly from the actual subsurface conditions encountered during construction.

The earliest boring data provided comprised geologic sections with the materials encountered shown in graphic form. Numerical data relating to sampler penetration resistance, rock sampling technique, rock core recoveries or Rock Quality Designation (RQD) were not always available. Boring locations were also questionable. The general subsurface profile, comprises up to 12m of miscellaneous fill, 2 to 11m of organic material, thin isolated layers of sand, and glacial till ranging from about 4 to 7m thick, where present. Bedrock underlies the till in the western portion of the site and underlies the organic layer along the east wall.

Some of the early boring data indicated substantial thicknesses of “soft rock,” particularly in the southwest corner of the site. Absent the detailed numerical data or details of the drilling process, the rock quality descriptions were in doubt. Top of rock contours developed from the available information varied from Elev. -6m in the northwest corner of the site to Elev. -12.2m at a location along the south property line about 1/3 from the southwest corner and about 1/3 from the east wall in the center of the site.

A preliminary subsurface investigation comprising 10 borings was performed in late Fall 1999. The ten borings were spread out over Sites 3 and 4 and two additional adjacent sites to provide subsurface information in areas not previously explored as well as to verify at least some of the available data. Even amongst the available data there were apparent conflicts. Near the center of Site 4 where the tower was ultimately to be located, two available borings were within a few meters of each other. The 1977 boring designated D-1, encountered about 0.3m of decomposed rock whereas the historical boring, L-8 showed “soft” rock more than 5m thick. As there were no sampling data available from Boring L-8, we concluded that the rock had probably been drilled rather than cored and sampled and its quality was easily misinterpreted.
Our Boring No. B-7 in the southwest corner of the site encountered only 0.6 meters of decomposed rock whereas the nearby earlier boring data indicated as much as 8m of decomposed rock. Contours of the top of rock developed from the 10 preliminary borings were similar to the contours developed from the available information borings except that a depression in the surface of rock in the center of the site was about 1.5 meters deeper than was previously believed to exist and the deep zone of rock at Elev. –12.2m along the south wall covered a broader area than had previously been found. For the most part, the thickness of the decomposed rock encountered in the 10 preliminary borings was limited to 0.6m or less.

The final subsurface investigation comprised 22 additional borings in Sites 3 and 4 to provide uniform coverage and satisfy code requirements on the number of borings. Bedrock contours developed from the final borings did not change substantially from the preliminary subsurface investigation but did show a somewhat broader area for the portion of the site where bedrock was deepest. Maximum thickness of decomposed or weathered rock in the final borings was 2m, in two borings, but was typically less than 0.6m.

Rock quality and allowable bearing for footings supported at the top of rock were evaluated based on rock core recovery and Rock Quality Designation. RQD is the sum of the lengths of recovered rock core fragments 100mm or longer between natural breaks divided by the length of core run expressed as percent. Core recoveries, defined as the total length of core recovered divided by the length of core runs, were typically in excess of 80 percent and RQD was typically about 75 percent. We judged that the allowable bearing intensity at the top of rock would be 3.8MPa and that in some areas the allowable bearing intensity could increase to as much as 5.7MPa. Allowable bearing could also increase with embedment beyond 300mm to a maximum increase of 100 percent at a depth of 3.4m embedment. The thin layers of weathered and/or decomposed rock encountered were to be excavated as the allowable bearing was limited to no more than 0.8MPa.

**ALTERNATIVE FOUNDATION SCHEMES**

Three foundation schemes were initially studied for development of the former industrial site. The first consisted of a diaphragm wall perimeter, a four floor deep basement and the superstructure supported on piers to rock. Column loads ranged up to 76 MN in the tower core. This scheme permitted the placement of all the parking and unloading facilities below grade but required the complete removal of 185,000 cubic meters of soil and 19,000 cubic meters of rock. 80,000 cubic meters of the excavated soil was contaminated by industrial byproducts and had to be removed and disposed of at a special containment site. This scheme yields a totally clean site, and was considered to be an enhancement of the site and a long-term benefit. The highly disadvantageous ground conditions at the site; river bottom mud, glacial till, boulders, decomposed rock, abandoned piers, riprap filled, timber crib bulkhead walls and a high ground water level dictated the use of a diaphragm wall, although an outside party suggested that driven steel sheeting could be successfully installed! Up to that time, and because subsurface conditions are generally similar along much of the Jersey City waterfront, no significant basements had ever been constructed in that area. We needed to convince the new Owners of the advantages of diaphragm wall construction and needed to instruct them on the technology. Ironically, we used the World Trade Center as an example of methodology. It was not until two years later that “slurry wall” and “bathtub” became household phrases.

The second scheme consisted of a shallow, one basement excavation temporarily supported by sheeting and New York City “caissons” (drilled in piles with steel cores) proposed to support the superstructure. This scheme had the advantage of low cost support of excavation but was burdened by the high cost of caissons that had to be installed through the forest of existing 560mm diameter piling left after the demolition of the former industrial buildings and the permanent entombment of a significant portion of the contaminated ground. Permanent entombment was undesirable because of the long-term liability associated with a contaminated property. Additionally, a majority of the parking spaces would have had to be sandwiched between the lobby and the office floors above. In retrospect, the caissons would have been even more costly when considering the additional depths necessary to obtain suitable rock throughout the site.

The third scheme consisted of the installation of caissons for support of the superstructure and construction of a platform at grade. With this scheme, all of the contaminated soil would have been entombed below the at-grade floor and all of the parking and unloading facilities would have been located above grade.

Although modestly more costly and potentially longer to construct, it was concluded that the diaphragm wall scheme offered the best operational and aesthetic scheme and the best long-term value.
**DIAPHRAGM WALL**

Because of the significant variations in the site geologic conditions, two different diaphragm wall sections were developed. Along the east and south the design required the installation of a full depth 800mm thick diaphragm wall, embedded 600mm into rock, supported by four tiers of tieback anchors set in the rock. (Fig.3)

Fig. 3 - Section at East Wall

Along the west and north, the design utilized a 600mm thick “hanging wall” embedded into the rock a minimum of 600mm but of sufficient depth to always place the key below the structural slab for the third basement level (fig. 4). Only three levels of anchors were required at these locations, however, the wall was pinned to the rock at the bottom with nominally prestressed rock anchors prior to formation of the rock shelf. Because of anticipated unfavorable rock conditions, the rock shelf was reinforced with closely spaced rock bolts, supplemented with additional bolts as poorer rock was exposed during general excavation and as the result of some over-blasting. The contractor elected to utilize an 800mm thick wall for the full perimeter. Panels were originally designed at widths of 6m but at the Contractor’s request were redesigned to 7.8m. Panel depths ranged from about 11 to 18m.

Diaphragm wall construction and basement excavation work was performed by the joint venture of DFI Corporate Members, E. E. Cruz Company and Nicholson Construction Company. Foundations were constructed by E. E. Cruz. The diaphragm wall work was performed using two set-ups and crews.

The presence of old foundations and debris in the fill (Fig. 5) necessitated pretrenching to a depth of 3m by backhoe in advance of guidewall construction. Backfill in the pretrench consisted of a low strength concrete/flyash flowable fill. Hydraulic clamshell buckets were used for panel excavation and a variety of drop hammers were used to chisel the bedrock. A total of 71 panels were excavated and poured between August 2000 and January 2001. Excavation spoil was transported by truck to barges for final disposal. Average progress, including rock removal and desanding, was about 10.5 square meters per crew shift.

The new Hudson-Bergen Light Railroad was located only 4m from the west wall and was placed in operation at the start of work on the diaphragm wall, necessitating special care in the removal of abandoned piles along the alignment of the wall as well as with the installation of the wall and anchors.

Fig. 4 - Section at West Wall

![Fig. 4 - Section at West Wall](image)

Fig. 5 - Existing piles before removal at South Wall

![Fig. 5 - Existing piles before removal at South Wall](image)
Catenary poles were braced at their tops to prevent rotation and sagging of pantograph cables and the shallow caissons supporting the poles were underpinned. The 490 prestressed tieback anchor capacities varied from 600kN to 3000kN. The anchors were intended to be detensioned once the permanent floors were in place, however, as a response to the September 11, 2001 attack the owner changed the scope of the project. Fortunately, slope inclinometers had been set in several panels around the site to permit monitoring of deflections during excavation. When the owner delayed completion of the north garage by a year or more, the inclinometers proved useful in demonstrating that wall movement was not occurring. The north garage was completed in mid-2003 and the tiebacks detensioned. Horizontal wall movements due to wall excavation were in the order 50 to 75mm. Vibrations resulting from the destruction of the World Trade Center towers were reportedly the equivalent of a 2.2 earthquake. No noticeable damage to the then exposed diaphragm wall was observed.

The wall was analyzed in the conventional manner as a continuous beam and was reinforced for the maximum shears, moments and reactions.

FOUNDATION ELEMENTS

The initial design of tower foundations contemplated piers cast on rock at a design capacity of 3.8MPa and the installation of prestressed tiedown anchors for piers in the tower core area. With the discovery of poor rock conditions at subgrade it became necessary to redesign 26 piers in the southwest quadrant of the site to accommodate a reduced allowable bearing pressure 1.9MPa. At 21 other locations the piers were redesigned for support on 2200kN mini piles. The redesign efforts were initiated as the subgrade rock was exposed and the poor rock conditions identified.

ROCK VARIABILITY

During construction of the diaphragm wall, which required nominal embedment of 600mm, significant variation in the top of rock was encountered, particularly in the southwest corner. An unexpected deep seam of decomposed rock was encountered which necessitated the deepening of several panels by as much as 3m. As a result of that seam, 8 footings along the south wall were redesigned for support on 325mm diameter drilled minipiles rather than the originally anticipated footings. Numerous obstructions were encountered along the east wall where the diaphragm wall penetrated the old timber crib bulkhead. Panel depths along the east wall were about as expected. When the interior excavation reached roughly -11m and bedrock was encountered, a deep seam of decomposed rock was exposed stretching from about a third of the way south from the northeast corner along the east wall to the southwest corner of the site. This seam varied in thickness from less than 1m at some locations to 7m across and up to 7m deep in the northwest corner of the tower core. The decomposed rock was judged to have nominal bearing value of 0.8MPa and was unsuitable for support of the heavy footing loads.

Mass excavation in rock in the core area was by ripping, hoe ram, or drill and blast where rock quality was quite good. The decomposed rock could be easily removed by backhoe and/or hand operated mechanical equipment. Two major areas were affected by the discovery of the decomposed rock seam; the northwest corner of the tower core and the first line of column footings along the northern third of the east wall. The tower core was more dramatic. The deep seam of decomposed rock was excavated to as much as 7m below the mat and replaced with nearly 300 cubic meters of concrete (Fig. 6). Footings along the east wall were redesigned for support on drilled mini-piles. In most locations the rock immediately adjacent to the weathered seam is of 3.8MPa quality. The soft seam occasionally dipped at a steep angle below better quality rock. The narrowness of the seam and the high quality of the adjacent rock observed in the field made it apparent that borings not specifically encountering this weathered zone could not have recognized that this material was present.

Careful attention was paid to the corrosion protected tiedown anchors for the core and individual footings requiring anchors that they were drilled into the sound bedrock. Tower core anchors consisted of 63mm high strength bars at capacities up to 2100kN. Footings in the northern portion of the site requiring tiedown anchors had 44mm diameter high strength bars at nominal capacities of 1100kN grouted into the rock and prestressed.
CONCLUSIONS

The presence of the decomposed rock necessitated changes in the field to modify the allowable bearing of the specific footings and the redesign of footings for support on drilled mini-piles. Although there were indications of significant thicknesses of weathered rock in some of the early available boring data, the design subsurface investigation did not encounter this material despite the fact that borings were made very close to old borings and in locations between old borings, without encountering the decomposed rock. Exposure of the surface of rock in the mass excavation made the identification of these poorer materials easy and modifications to the design of foundations were made in a timely fashion. The presence of the decomposed seam could not have been predicted without actually having borings directly encountered by the material.

Fortunately, the decomposed rock was very dense and excavation into the deep seam, even relatively close to the perimeter diaphragm wall, did not result in significant seepage into the site. Had the seam not been exposed and the design modified, it is conceivable that foundations supported on rock drilled in the blind from above could have resulted in substantial problems to the building foundation. In the summer of 2001, construction of the superstructure began and topped out in 2002. Partial occupancy began in 2003. 20,000 cubic meters of water was removed from the site and treated prior to disposal. That total was remarkably low considering that it included all of the water inside the diaphragm wall, precipitation during the 14 month construction period, drill water used to install tieback anchors and seepage through the wall and rock. Infiltration quantities were negligible when compared to quantities contained within the site prior to excavation and quantities from precipitation.

This project was awarded the DFI 2003 Outstanding Project Award.

At the Annual Banquet in Miami, FL, Mueser Rutledge Consulting Engineers received the OPA and E. E. Cruz Company and Nicholson Construction Company received Team Recognition Awards.

Evolution of DFI Magazine
By Manuel A. (Manny) Fine, Managing Editor

In case you haven’t noticed, the evolution of the DFI publication from a newsletter to a full-featured magazine has been completed. Commencing with the last issue, Fall 2003, we have gone to full color. It will take a few issues before all of our repeat photos, etc. catch up with this transition, but we are now producing the DEEP FOUNDATIONS Magazine in full color. Our larger advertisers have, for the most part, embraced this change, realizing the added impact color can have in ads. We are seeing more and more of them supplying their advertising copy in color, taking advantage of the modest additional cost for color ads. We are attracting more advertising and the magazine is now generally self-sustaining when it comes to cost.

I used to worry about getting enough material to fill the publication. Now I get more than I can use. It has been exciting for me to be part of the transition from a modest newsletter to a real magazine. While the job of Managing Editor has its share of frustrations, it is a labor of love for me, allowing me to remain involved with the Industry in my “retirement” years.

I want to thank the many people who have gone to the trouble of telling me verbally, by e-mail and by fax, how much they look forward to each issue and how they enjoy the magazine. Your compliments are very gratifying and I appreciate your encouragement. I hope that you enjoy reading the Winter 2004 issue of DEEP FOUNDATIONS.

“When I am working on a problem I never think about beauty. I only think about how to solve the problem. But when I have finished, if the solution is not beautiful, I know it is wrong.”

Buckminster Fuller
30 Hudson Street, Jersey City, NJ
Steelwork for Office Tower emerging from deep basement excavation