“Value Engineering…?” – Changes During Construction

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ABSTRACT

"Value Engineering" is a frequently found clause in Construction contracts in the USA, allowing contractor-initiated design changes. Misleading is the interpretation of "value engineering" to imply cost savings shared with the owner, and its implementation during construction, is problematic. It is not surprising that such a clause would simply be ignored because it involves changes in design, often major changes in very short time; and change is feared and vehemently resisted by all parties, owner, designer, and contractor. The problem may lie in the divergence and separation of the designer/engineer and builder/contractor; their priorities and incentives are very different. The engineer spends years, even decades, in design and prepares contract documents often without a deep-seated understanding of construction methods, including geotechnical construction. Even worse, given extensive computational advancements, the designer submits exaggerated code-based designs with excessive safety factors. As for the contractor, he often builds without full appreciation of design principles or regard for design engineers. Owner budget and schedule constraints (not commensurate with his demands) and the ever-increasing litigious climate have exacerbated the situation. Adverse and hostile relationship between the various groups is often the norm with extended disputes and claims, not to mention the costs these entail. Redesign to apply a new technology or optimization of an inferior design just before construction becomes unthinkable. Four case histories are presented.

INTRODUCTION

Four case histories (spanning from 1998 to 2002) are presented, where major design changes were implemented in record speed during construction of several very large projects in the metropolitan New York area. They include, two Design/Build projects: **Case History 1)** the redesign of large diameter drilled shafts for the Hudson-Bergen Light Rail Transit System ($1.1 billion total value and $343 million for the Initial Operating Segment for this case); **Case History 2)** the elimination of deep caissons in favor of spread footings for a new $90 million MTA bus depot in Manhattan; and, two conventional Design/Bid/Build projects: **Case History 3)** maintaining in lieu of removal of a 100-year old abutment of the $72 million Queens Boulevard Bridge Replacement; **Case History 4)** the complete redesign of major retaining walls and actual use, for the first time outside Asia, of the "Giken" tubular pressed-in pipe piles as very high cantilever retaining walls, for the $150 million expansion of the Long Island Expressway.

Before describing the four case histories, it is noted that projects of such large magnitude involve armies of people with different backgrounds. For many, their basis of experience in design and/or construction is largely to follow codes and specifications and paper tracking has become the occupation for many more. Most of the parties involved in large projects do not feel the need nor have the incentive to consider cost-effective solutions. Often, they are not even aware that changes are necessary or possible. Moreover, in a litigious society such as in the USA, liability concerns impede innovation, much less implementing design changes once construction has begun. In their attempt to be "safe", many do not advocate or employ the very advances that are presented and discussed in journal articles and conferences. Owner budget constraints — not commensurate with his demands — and fast-track schedules — often unrealistic — have exacerbated the situation.

It is perhaps helpful to mention that the "valuable engineering" design changes for the cases described herein are the personal account of a geotechnical engineer "defecting" to the construction side (where a possibility for adequate change could be detected), combined with the least welcome attitude of a female passion and insistence. The process of change was unconventional, painful, and even comical.

CASE HISTORY 1 – Hudson-Bergen Light Rail Transit System, New Jersey:
"Redesign of Newport Viaduct Drilled Shaft Foundations"

The Project

The Hudson-Bergen Light Rail Transit System is a 33 Km Design-Build-Operate-Maintain light rail project in northern New Jersey. The Initial 16 Km of the project extends through Jersey City and includes a 0.8 Km long viaduct, a multiple-span bridge carrying the light rail in a north-south direction just west of the Hudson River. Near the mid-length, the bridge crosses over the entrance to Holland Tunnel leading to Manhattan, New York.

For the multi-span Newport viaduct, deep foundations including drilled shafts (bored piles) were selected with loads ranging from 4,450 KN to 16,300 KN (500 to 1,800 tons) from each pier. (In general, the design/build team followed the original bid reference documents including project design criteria, as well as mandated codes and specifications.) The initial design required the drilled shafts to extend through a thick zone of "completely weathered rock" and then be socketed into "sound" sandstone. The assigned design parameters were generally consistent with the bid reference documents, a unit end bearing resistance for rock of 0.8 MPa (8 tsf), increased to 1.2 MPa (12 tsf) after minimum 3 m penetration into "sound" bedrock. For "sound" rock, an allowable unit shaft resistance of 275 KPa (2.9 tsf) was specified.

The subsurface conditions along the bridge alignment consist of an upper 6 m thick granular fill over 3 m soft marine clay, underlain by 12 m of medium dense to very dense glacial deposits comprised of alternating layers of silty sands with gravel to clayey silts. Weathered sandstone, with an average thickness of about 7 m and described in the test boring logs as "completely weathered rock", extends below the glacial deposits with SPT-N values of 100 blows for only 25 mm to 150 mm penetration. Sandstone bedrock exists below a depth of about 28 m. The groundwater table at this site is shallow and within 3 m of the existing ground surface (Fig. 1.1).

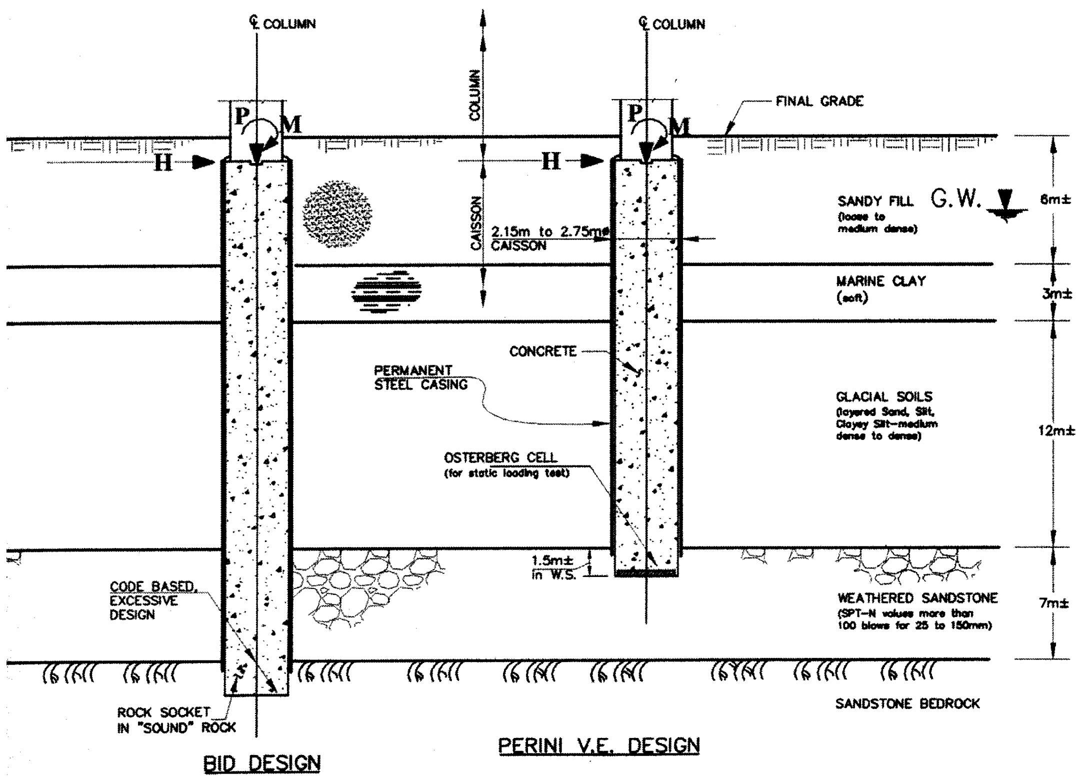

Fig 1.1. Typical Subsurface Section with Drilled Shafts

The Change

The initial design for the drilled shaft foundations supporting the viaduct was questioned by this author, given the highly variable nature of the sandstone rock and the implications of searching for "sound" sandstone at great depths. Encouraged by recent studies of intermediate geomaterials (O'Neill et al., 1996), it was deemed unnecessarily conservative to bypass the weathered sandstone in search of deeper "sound" rock. Also, a report (Baker, 1988) on foundations for an adjacent 30-story office tower supported on 1.2 m diameter drilled shafts in the "completely weathered rock" zone showed resistances that were substantially higher than what was specified for "sound" rock. The report contained two conventional head-down static loading tests that measured unit toe resistance ranging from 7.8 MPa (80 tsf) @ 35 mm movement, to 10.4 MPa (108 tsf) @ 15 mm movement. The higher unit toe resistance was measured in the upper parts of the weathered layer having SPT N-indices of 100 for only 150 mm penetration. From the same loading tests, a unit shaft resistance of 2.2 MPa (23 tsf) had been deduced for the weathered material.

With the above and other design information at hand, the initial simplistic design was rejected and higher design values for the weathered rock were implemented subject to verification in full-scale static loading tests. The intent of the testing was to demonstrate that the drilled shafts can be supported on top of or just within the "Weathered Sandstone", believed to provide adequate shaft and toe resistances to support the bridge column loads. More important, the testing was also intended to evaluate the impact of the proposed construction procedures on the axial capacity of the drilled shafts. Because of the large design loads, the Osterberg-cell (O-cell) test method was selected.

It is perhaps not a surprise that the above change was strongly resisted by all parties involved, including the designers, whose "safe" design in "sound" rock was challenged. What was not anticipated, at least by the author, was the drilled-shaft subcontractor's reluctance to accept "reduced drilling quantities". The first loading test with the O-cell was not successful because of premature mobilization of shaft resistance caused by excessive drilling disturbances. However, after making all the necessary adjustments/refinement in the construction procedures, the second loading test, which immediately followed, achieved the intended "valuable" engineering change, as described below.

<u>Testing Program</u>

Principles of the O-cell method can be found elsewhere (Osterberg, 1994; Schmertmann, 1997; Fellenius, 2001).

In summary, the testing is conducted from bottom up with the use of a sacrificial hydraulic jack — i.e. the Osterberg Cell, placed near the base of the shaft to be tested. As the O-cell expands, it pushes the shaft upward and the base downward. Unlike the classic head-down static loading test, the O-cell allows the separate measurements of load-movement behavior of the shaft and the base. The upward load movement is governed by the shear resistance characteristics of the soil or rock along the shaft, whereas the downward load movement is governed by the compressibility of the soil or rock below the shaft toe (Fellenius et al., 1999).

For this project, the O-cell loading test was conducted on one of the central (2.15 m in diameter) drilled shafts, which was prepared as a production caisson. The subsurface profile at the test location, as determined from a nearby test boring and as observed from the drill cuttings, consisted of 4.6 m thick granular fill, over 5.5 m marine clay, underlain by 10.8 m glacial deposits of silty sands with gravel. What had been described in a nearby boring as "completely weathered rock" at about 21 m depth below the existing grade, was recovered as a 1 m diameter solid sandstone core (Photo 1.1), immediately underlain by soil-like, completely weathered rock. The test shaft was advanced about 3 m into the weathered sandstone. The total length of the test shaft was about 24 m below the ground surface.

Photo 1.1 - Recovered 1m diameter Core from the "Completely Weathered Rock" zone

The test shaft (as well as all the remaining 44 production shafts with diameters ranging from 2.15 m to 2.75 m) was constructed using the slurry method to maintain a stable hole. Drilling began with soil augers to 1.5 m depth before introducing the polymer slurry, followed by further drilling and installing a slightly oversized 10 m long temporary casing to support the upper fill and marine clay. Both soil and rock augers were used to advance the shaft below the temporary casing and through the glacial soils. The slurry level inside the shaft was maintained approximately 1.5 m above the outside groundwater level, just below the existing ground surface.

Once weathered rock was reached, a permanent steel casing (Photo 1.2) with welded teeth was inserted into the shaft and twisted for about 0.3 m into the weathered sandstone. Drilling below the permanent casing was then continued at slightly reduced 2 m diameter using rock augers, sometimes assisted with core barrels to core the harder rock. For the test shaft, the length of the socket extending into the weathered sandstone was 2.5 m. The sides of the socket were scraped to remove softened material. At the end of drilling, the bottom of the socket was cleared of cuttings and accumulated sediments were removed using a clean-out bucket. It was estimated that approximately 25 mm of sediments remained at the base of the test shaft before beginning the O-cell static loading test.

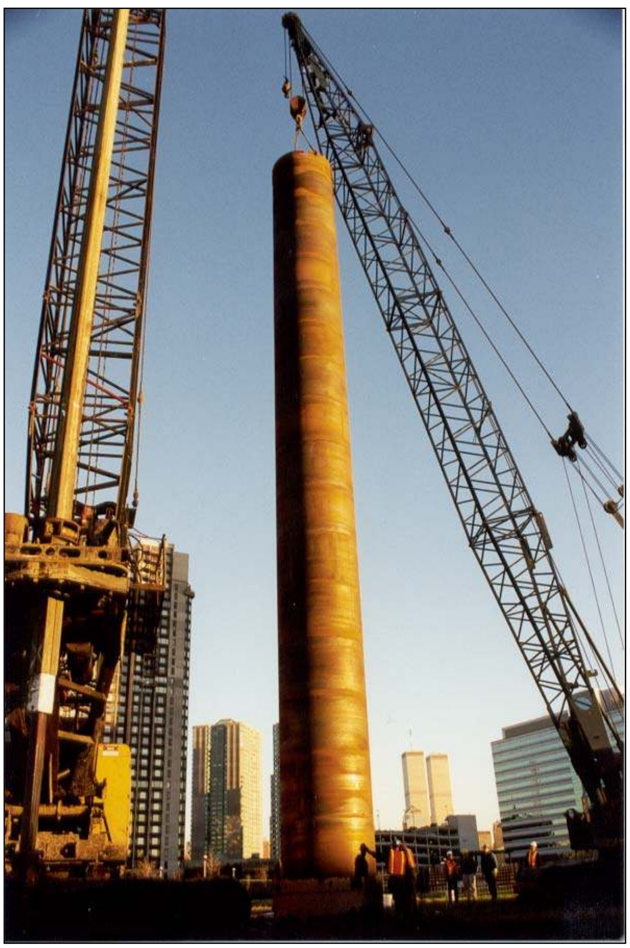

Photo 1.2. – Installation of Permanent Steel Casing

Following drilling and clean-out of the test shaft, three 533 mm (21 inch) diameter sacrificial O-cells (welded and contained between two circular steel plates, and mounted on a steel frame), were lowered to the bottom of the test shaft (Photos 1.3 and 1.4).

Photo 1.3. – Three O-Cells between two steel plates

Photo 1.4. – The O-Cell assembly, complete with Instrumentation being lowered to the bottom of the prepared Drilled Shaft (before concreting)

The three O-cells were capable of applying a total combined load of 34,200 KN (3,845 tons). The O-cell assembly was complete with instrumentation including sister bar vibrating wire strain gages at three levels along the shaft, to measure the shaft resistance (side shear load transfer) in the various layers.

Once the O-cell assembly was positioned inside the test shaft, concrete placement by pumping from the bottom up proceeded, until the level reached to within 2.2 m below the final head level. (The upper 2.2 m part of the shaft was later reinforced and concreted monolithic with the bridge pier. It is noted that the design optimization had also eliminated the use of full-length reinforcing cages; instead, the drilled shafts consisted of 16 mm thick permanent steel casing filled with 34.5 MPa (5,000 psi) concrete, and a steel reinforcement cage limited to the upper 2.2 m part of the shaft.)

Following placement of the concrete, the annulus between the outer temporary casing and the inner permanent casing in the top 10 m of the shaft was grouted via a tremie pipe and the temporary casing was gradually pulled as grout filled the void that was created during drilling.

The O-cell static loading test was performed by Loadtest Inc., on December 21, 1998, five days after placement of the shaft concrete and its attainment of the necessary compressive strength. The quick loading test procedure was followed and readings of all gages were obtained at 1, 2 and 4 minutes for each of the 14 loading increments. The three O-cells were pressurized to a total test load of 20,600 KN (2,315 tons) (half upward and half downward). The loading was halted when it was determined that the side shear in the overburden soils surrounding the permanent casing was approaching its full resistance mobilization.

Results

Based on the results of the O-cell static loading test, the total upward movement of the shaft was 4.6 mm at a maximum upward net load (gross load minus the buoyant weight of the shaft) of 9,300 KN (1,040 tons). The total downward movement of the shaft base at the maximum downward gross load (net load plus buoyant weight of shaft) of 10,300 KN (1160 tons) was 4.5 mm. This movement is only 0.2% of the shaft socket base diameter of 2 meter. Because of such small base movement and no apparent creep, it can be concluded that the ultimate end-bearing resistance of the shaft founded in the “completely weathered sandstone” was never reached.

The interpreted average unit bearing resistance in the weathered sandstone at the measured nominal 4.5 mm movement was 3.35 MPa (35 tsf). While not approaching its anticipated ultimate capacity, this deduced unit bearing resistance in the “completely weathered rock” was much higher than the bid design value, of 0.8 MPa to 1.2 MPa (8 to 12 tsf), specified for “sound” rock.

It is further noted that the unit shaft resistances measured during the loading with the O-cells, were lower than measured for the previously described conventional head-down static loading tests at the nearby office tower. This could be due to that, in a conventional head-down test the instrumentation will not measure the load locked-in the pile (residual load) before the start of the test. When the residual load is neglected, the shaft resistance is overestimated. The use of polymer slurry and running the test only five days after drilling and concreting could also have resulted in a lower range of shaft resistance values.

Table 1 below summarizes the results of the Side Shear Transfer data for the various subsurface layers along the test shaft from the strain gage instrumentation.

Table 1. Side Shear Transfer from O-Cell Strain Gage Data

Load Transfer Zone	**Unit Side Shear Resistance**
Grouted Zone outside Permanent Casing (upper ~ 9.3 m of overburden soils)	30 KPa (0.31 tsf)
Non-Grouted Zone outside Permanent Casing (~11.6 m glacial soils; drilled under slurry)	6 KPa (0.07 tsf)
Upper ~1.7 m Socket in Weathered Sandstone (includes 0.3m casing embedded into W.S.)	333 KPa (3.48 tsf)
Lower ~ 1.1 m Socket in Weathered Sandstone	302 KPa (3.16 tsf)

Figure 1.2, below, shows the Load vs. Movement curves, separately, for each of the upward and downward movement directions.

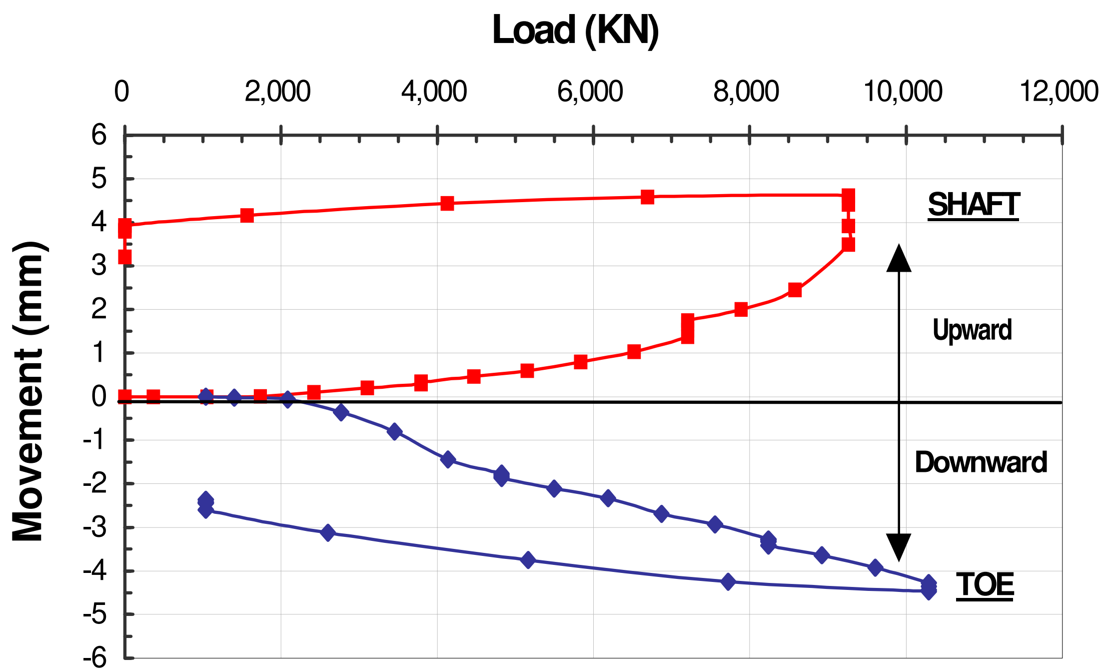

Fig. 1.2 –Load-Movement Curves from O-Cell Test Results

Conclusion

Despite the strong resistance to change by the parties involved, the main objective of demonstrating that the "Weathered Sandstone" should not be bypassed in search of "Sound" rock was achieved. The Newport Viaduct is now supported on large diameter drilled shafts with only nominal 1.5 m penetration or "sockets" in the "completely weathered rock" material.

The unnecessary search for "sound" sandstone at great depths was eliminated, together with unavoidable disputes and delays. In addition to considerable time and cost savings, the design change contributed to some valuable insight and experience in design and construction of large diameter drilled shafts in northern New Jersey.

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CASE HISTORY 2. – 100th Street Bus Depot, New York City: "Elimination of Deep Caissons in Favor of Spread Footings"

The Project

The 100th Street Bus Depot is a new five-story bus terminal for the New York City Metropolitan Transit Authority (MTA) occupying one city block in Manhattan between 99th and 100th Streets, and between Lexington and Park Avenues. It replaces a former two-story garage, built in the 1890's and initially used as a trolley-car barn. The new structure is a steel-framed building with concrete floors and includes a partial basement near its middle. The easternmost column line with design loads of 10 MN (1,125 tons) is within 1.5 m of the underlying multi-tube Lexington Avenue subway system.

To support the new columns along the subway line, the project bid plans had called for 1 m diameter and 17 m deep caissons. The upper 14 m of each caisson unit would have required 50 mm isolation from the surrounding rock thus transferring the column loads to below the base of the existing tunnel, via 3 m long sockets in bedrock. (This follows a routine requirement by MTA to prevent stress from being imposed on the roof and walls of their tunnels.)

The bid documents contained extensive test boring information, which revealed that the project site was underlain by massive Manhattan mica Schist bedrock within just 3 m below street level. In fact, the Lexington Avenue two-level and multi-tube subway had been tunneled through this rock circa 1910, leaving an about 6 m thick solid rock roof over the upper-level subway (Fig. 2). The NX rock core recovery values were near 100 % with an average RQD of 75 %.

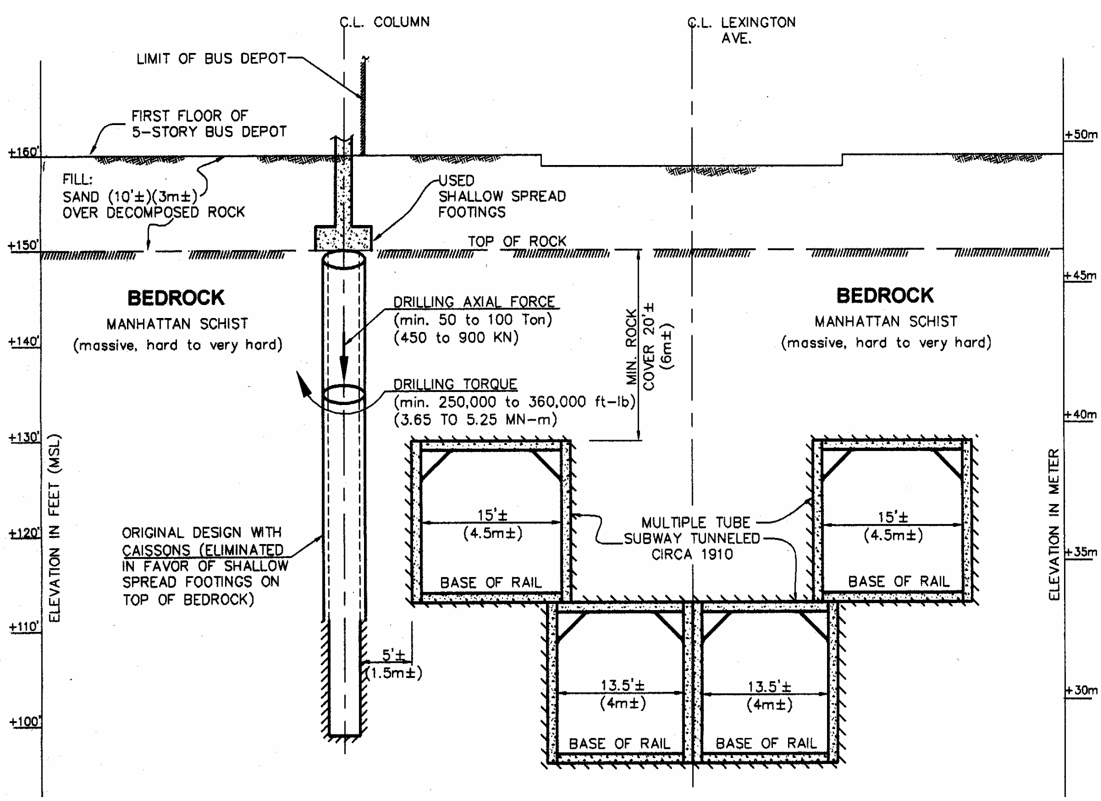

Fig. 2 – Typical Subsurface Section at Bus Depot

The Change

The Bus Depot project was let as a design/build contract with a very aggressive fast-track schedule. Both the pre-bid preliminary design and the subsequent final design by the Contractor's own hired design engineers relied heavily on very restrictive codes, including unnecessarily conservative approaches to seismic design. Any suggestion for value engineering change was undesirable and was strongly resisted. Yet, objecting to the blind reliance on codes in foundation design, the author questioned the real need for the deep caissons to support the columns along the subway. The unsupported statement that no stresses from the new structure can be imposed on the roof and walls of the adjacent tunnel was not sufficient reason for a deep foundation design.

Unconvinced that deep shafts were necessary, the author rushed to MTA's warehouse where many rows of the project rock core boxes were neatly stacked. Careful inspection and some simple testing of the rock cores with the blows of a geologic hammer gave support to the reservations about the design. Later, as expected, the schist bedrock at this site was found to be hard to very hard with unconfined compressive strength values estimated to range from at least 70 MPa to over 100 MPa (10,000 psi to 15,000 psi). The tensile strength of the rock was assessed to be minimum 5 MPa (750 psi).

It may be necessary to mention here that prior to the bid, more than 40 deep test borings including extensive coring of the rock had been carried out. However, not a single test was conducted on the cores to determine the strength of the rock. It is discouraging to realize that in this and in many other projects, test borings are performed to simply satisfy code requirements. Then, the boring logs become just part of the bid package for the contractor to review and, in essence, to become responsible and liable for the subsurface conditions encountered. It is further noted that contract specifications including this project, often demand requirements such as "no damage", "no movement" or "no vibration", thereby shifting all liability of underground work onto the contractor.

It is not surprising that such shifting of liability has resulted in unwarranted contingency for the contractor and is a source of claims and disputes. The need for careful evaluation of all aspects of a project, from inception to construction completion, including the very often-neglected structure/foundation interaction could perhaps alleviate some pain during construction, where the real test for any design begins.

Results

Convinced that deep foundations were not warranted for the 100th Street Bus Depot because good hard rock was so shallow, the fight for using shallow footings instead, began. At the pre-construction meeting, the author rolled a sample of the drilled rock core onto the conference table to show the more than 25 attendees what actually lay beneath the surface. An unrelenting argument was presented, dramatizing the nominal stresses on the roof and walls of the tunnel induced by shallow spread footings supported only on top of bedrock, as opposed to the substantial stresses caused due to drilling of the specified deep shafts so very near the tunnel (Fig. 2). To overcome the tensile strength and actually cut the hard rock, by drilling or coring, large axial forces and a substantial torque from the drilling machine would be required, it was argued. These drilling induced stresses immediately adjacent to the tunnel would be at least ten times larger than the maximum stresses induced from a shallow spread footing, under extreme loading condition, including the code-based unrealistically large seismic loads. It was, therefore, clear that the strict limitations imposed by the bid regarding impact on the adjacent tunnel were not realistic, nor were they consistent.

Conclusion

Eventually, the deep caissons were eliminated and shallow spread footings on top of the mica schist bedrock were used to support the columns of the new structure.

CASE HISTORY 3 – Reconstruction of Queens Boulevard Bridge, New York: "Saving the100-year old South Abutment"

The Project

The reconstruction of the 100-year old Queens Boulevard Bridge (QBB) for the New York City Department of Transportation was let as a conventional design-bid-build contract with a very aggressive schedule. The QBB is a major bridge crossing Sunnyside railroad yard where an extensive network of railway tracks is in constant use. Immediately above the QBB runs the New York City Transit Authority "elevated subway" line, leading to the underground subway system in Manhattan. The QBB itself is situated in one of the most congested areas of Queens, NY. This, combined with an active railway system 7 m below the bridge, and an active train system less than 9 m above the bridge, can make any construction activity in the middle a very difficult task. These physical constraints are further complicated because different agencies own and operate the various infrastructures.

The contract bid plans had specified the complete removal and replacement of the bridge superstructure and the complete removal and replacement of only one of the abutments, the South Abutment. The foundations of all the existing 18 intermediate piers and the north abutment were to be maintained.
The existing south abutment is a 30 m long concrete gravity structure with shallow spread footing (Fig. 3a). The north abutment and the intermediate piers are supported on timber piles.

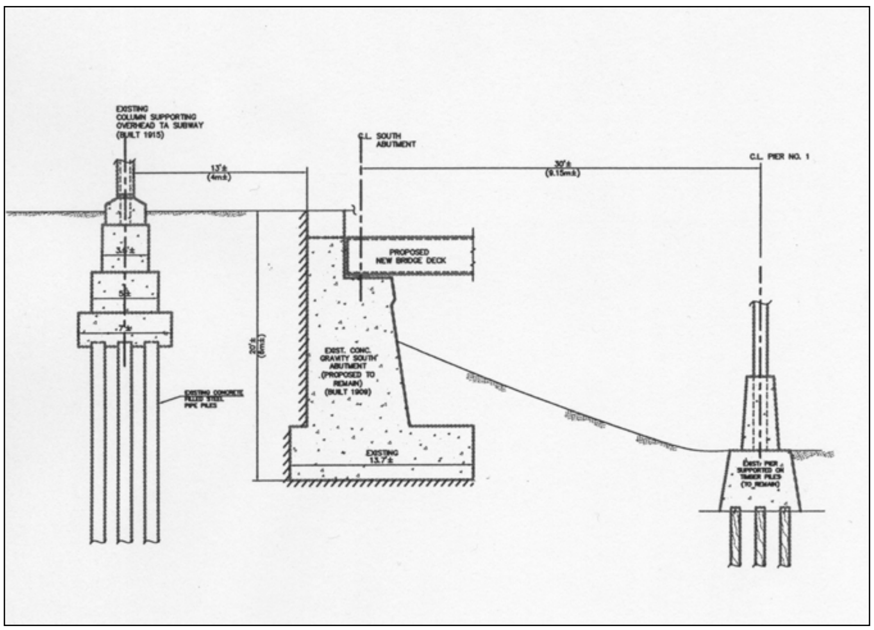

Fig. 3a – Typical Section at South Abutment of Queens Boulevard Bridge

The contract had called for replacing the existing south abutment with a new reinforced concrete structure, supported on deep concrete-filled steel pipe piles. For removal and replacement of the existing abutment, the bid plans had specified an excavation support system comprised of soldier piles / lagging, with tiebacks. This excavation support system was in very close proximity to the deep foundations of the existing overhead "elevated subway" bents, built circa 1915.

The subsurface conditions at the 100-year old railroad yard within which all the foundations of QBB are founded, are comprised of an upper 8 m thick silty sand fill, overlying a 1.5 m thin layer of organic clayey silt with peat, over 15 m thick dense glacial deposits of silty sands and gravel, over schist bedrock (Fig. 3b).

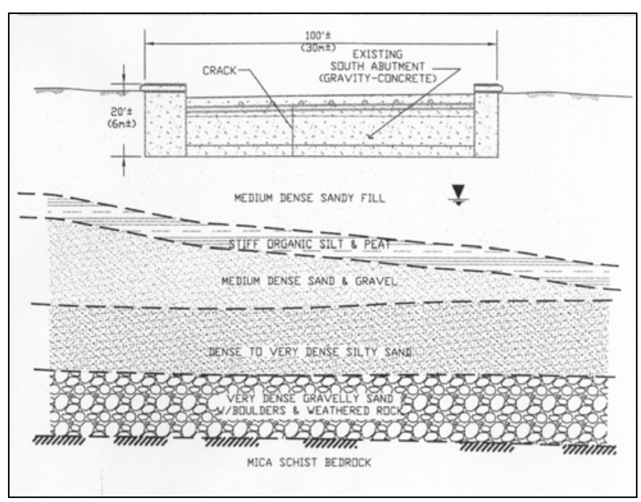

Fig. 3b – Typical Subsurface Profile at the South Abutment

The Change

Upon award of the construction contract, finding the reason behind the complete removal and replacement of only one of the twenty supporting elements of the new lighter superstructure of the QBB began. The complexity of the site and the very real potential for conflicts between such closely situated structures administered by so many different agencies provided ample reason to question the bid design. Indeed, except for one obvious crack, which was easily repairable, there did not appear to be much justification for the complete removal of the south abutment.

Value engineering to maintain this gravity abutment with its shallow spread footing (in lieu of a new pile supported structure) was initiated, but it received an unbelievably fierce resistance. Unfortunately, this is an only too-common reaction whenever spread footings in lieu of deep foundations are proposed. Very often, deep foundations are selected without proper evaluation of constructability or impact due to construction. In many situations, shallow footings perform as well, if not better than deep foundations, yet they are consistently ignored. More recently, the excuse of "seismic consideration" is immediately presented without real understanding or evaluation of dynamic behavior.

Despite the obstacles, however, saving the historic structure became a personal mission for the author. Extensive settlement, and static, dynamic, and liquefaction analyses were performed in support of maintaining it.

Results

The record drawings of the existing South Abutment contained a valuable note indicating that the massive concrete gravity structure had experienced about 75 mm settlement during construction from May 1909 to October 1910. This was not unusual, because the abutment is underlain by a sandy fill layer, which probably was in a loose state some 100 years ago. (It is likely that the subsequent use of timber piles to support the remaining piers and the north abutment was a result of the observed movement.)

Since the bridge (including the south abutment) had remained in successful service for over 90 years, further settlements, if any must have been small. The probable cause for the 10 mm wide vertical crack in the abutment was differential settlement resulting from consolidation of the thin organic layer below the fill. However, after nearly 100 years, the structure had reached a complete state of equilibrium. Furthermore, because the new loads from the replacement superstructure would be lighter than the one removed, no further settlements were to be anticipated. It is noted that the net bearing stress on the foundation soils imposed by the existing or the new bridge do not exceed a nominal 100 KPa (1 tsf).

The seismic analysis of the abutment proved to be very contentious. Efforts were not spared by the parties involved to impede the consistent conclusion that the existing abutment had adequate dynamic resistance. Even with unrealistic peak ground acceleration, and excessive seismic loads used in the extensive analyses (much higher than that assumed for the other elements of the same bridge), the dynamic performance of the abutment was still more than adequate.

The "drama" of soil liquefaction and the suggestion that 1.2 m settlements would occur due to liquefaction, was even more unrealistic. Response analysis revealed that such settlements, if any would not exceed 25 mm under a "fictitious" Magnitude 6.5 Earthquake in Queens, New York. Ironically, the "elevated subway" bents above the subject abutment cannot withstand an earthquake magnitude of 5.0 or 5.5, typically assumed for this site.

Moreover, the original bid design involving abutment removal had easily overlooked potential movements inherent with deep excavation and tieback installation, not to mention the vibrations from driving new piles, so near the vulnerable existing structures.

Conclusion

Needless to say, the South Abutment of the Queens Boulevard Bridge was saved.

CASE HISTORY 4 – Expansion of the Long Island Expressway, New York: "Redesign of Retaining Walls with "Giken" Tubular Pressed-in Steel Pipe Piles"

The Project

The expansion of the ever-congested Long Island Expressway (LIE) over Cross Island Parkway (CIP) just outside Manhattan, New York included numerous retaining walls and several deep cuts for bridge expansions. The bid plans for this conventional design/bid/build project contained complete designs for both permanent retaining walls and temporary excavation support systems.

The main specified retaining wall and support method consisted of very deep, 1 m diameter drilled-in soldier piles/lagging, complemented with tiebacks or bracing. Unusually heavy (over 1000 kg/m), and very long (25 m) H-beams with welded-on cover plates were specified. To install such beams, deep drilling in granular soils below the water table, would have been required

Soon after start of construction in the fall of 2000, serious supply problems for the specified heavy beams ensued. It became necessary to explore alternative systems for at least three major structures: a 78 m long permanent retaining wall SP-1 at maximum 8.5 m height; an excavation system to support an 11 m deep cut for construction of the new westerly abutment of the LIE bridge over CIP; and an excavation system to support a 9 m deep cut for the reconstruction of the abutments of Marathon Parkway Bridge over LIE.

The subsurface conditions at this site are characterized as terminal moraine glacial deposits consisting of medium dense, becoming very dense with depth, sands and gravel with varying proportions of silt, frequent cobbles, and numerous boulders. Groundwater is present just below the proposed excavation levels.

The Change

For this case, a value engineering change proposal was welcomed by the owner (New York State Department of Transportation, NYSDOT.) As usual, the fast track schedule of this major project and producing a new design during construction became a challenge, requiring the full and amicable cooperation of NYSDOT.

Interestingly, the beginning of redesign and the value engineering process for the LIE retaining walls coincided with an international conference on deep foundations in New York City on October 5 to 7, 2000. Agonizing over the strict limitations imposed by NYSDOT not only on soil parameters, but also on their requested design methodology based on codes, the author felt compelled to openly express her frustration with some of the attendees at that conference which apparently included the Giken "Press-in Method" representative. Thus, Giken responded to a request to provide *"not just video animation or colorful brochures, but good technical literature"*. Read and absorbed during that weekend, a presentation of the concept of the innovative technology was made to NYSDOT. In less than one week time following the conference, the complete redesign of one of the deep excavation support systems, in full cantilever, was submitted as value engineering change proposal. The installation of the permanent wall and the first phase of the two deep excavation support systems were completed in early July 2001. The second phase was completed in the spring of 2002. The Giken Press-in pipe piles system, a first outside the Far East, not only resolved the steel supply problem employing domestic lightweight pipe piles, but also allowed the use of the valuable soil-pile interaction design approach.

Design and Installation

The Giken "Press-in Method", also known as the "Silent Piler" (because it is virtually noiseless and vibration-free), is described elsewhere (Bearss et al. 2002; ENR 2001; White et al. 2000). It uses constant outside diameter (ranging from 800 mm to 1080 mm) open-ended steel pipe piles, which are pressed into the ground with a powerful hydraulic push piler. An initial "reaction stand" is required to install the first three piles. Thereafter, the piler literally rides over and grasps three piles, as it presses the next pile. (The reaction force to hydraulically press a subsequent pile is derived from the negative skin friction of previously installed piles.) The piles are nearly contiguous with maximum separation of 180 mm. This nominal gap is closed with welded-on Pipe-Tee (P-T) interlocks, or with flat bars. The pile installation is guided by laser beam resulting in remarkable small alignment deviation of less than 3 mm (1/8").

For this project, a constant 914 mm outside diameter open-ended steel pipe piles were pressed almost contiguously into the ground with a powerful 260 metric tonne capacity hydraulic push piler. The nominal gaps of 180 mm between adjacent piles were closed with shop-welded P-T interlocks in the case of the permanent wall, and with flat bars for the temporary support, extending a nominal 1.5 m below the excavation level. To aid in penetrating into the dense granular soils, two built-in water jets with maximum 7,000 KN/m^2 (1,000 psi) pressure per jet were used. The entire installation process was virtually without vibration or noise. The laser beam guided pile installation resulted in remarkable straight alignment.

Photos 4.1 and 4.2 below show the Giken piler during installation of the LIE retaining wall SP-1.

Photo 4.1. – Begin of Installation of Pile with Giken Piler

Photo 4.2. – Close up of Giken Piler with Pile

Results

The 0.914 m outside diameter steel pipe piles possess significant bending resistance to lateral loads and as such, support in full cantilever of the deep cuts, was achieved — the most desirable excavation support system for a contractor. Deflection based methods for the analysis of piles under lateral loads (Reese et al. 1974; API 1993; Reese et al. 2000; Reese and Van Impe 2001), were employed and a parametric analysis was performed with varying loading and subsurface conditions. The total length and wall thickness of each pile was selected based on maximum allowable deflection at the pile head and allowable bending stresses in the pile itself. Consistently, the results of the analysis indicated that a depth of embedment for each pile about equal to the cantilever height would provide an adequate performance. In general, thin-wall pipe piles offered a better flexible performance (Bedian 2002).

Upon final excavation, measured movements at the pile heads were remarkably close to the predicted values (based on estimated p-y curves for the granular soils at this site.)

Measured movements at the permanent wall were as per design, less than 25 mm.

For the 10.7 m deep west abutment excavation where large movements (150 mm to 180 mm) were allowed, the maximum measured pile top movements were as follows (Fig. 4.1): along the centerline of LIE where 25 mm thick wall tubular piles were pressed-in, the maximum pile-head movement was 125 mm. Along the back of the new abutment where 17.5 mm thick wall piles were pressed-in, the average measured pile head movement was 161 mm.

The total lateral pile head movements are inclusive of about 25 mm deflection experienced by each tubular pile immediately following the driving of the new abutment pile foundations (324 mm diameter steel pipe piles) at the bottom of the cut. The additional movement was probably due to the pile driving vibration-induced temporary loosening / liquefaction of the submerged sands below the excavation, upon removal of the 10.7 m overburden soils.

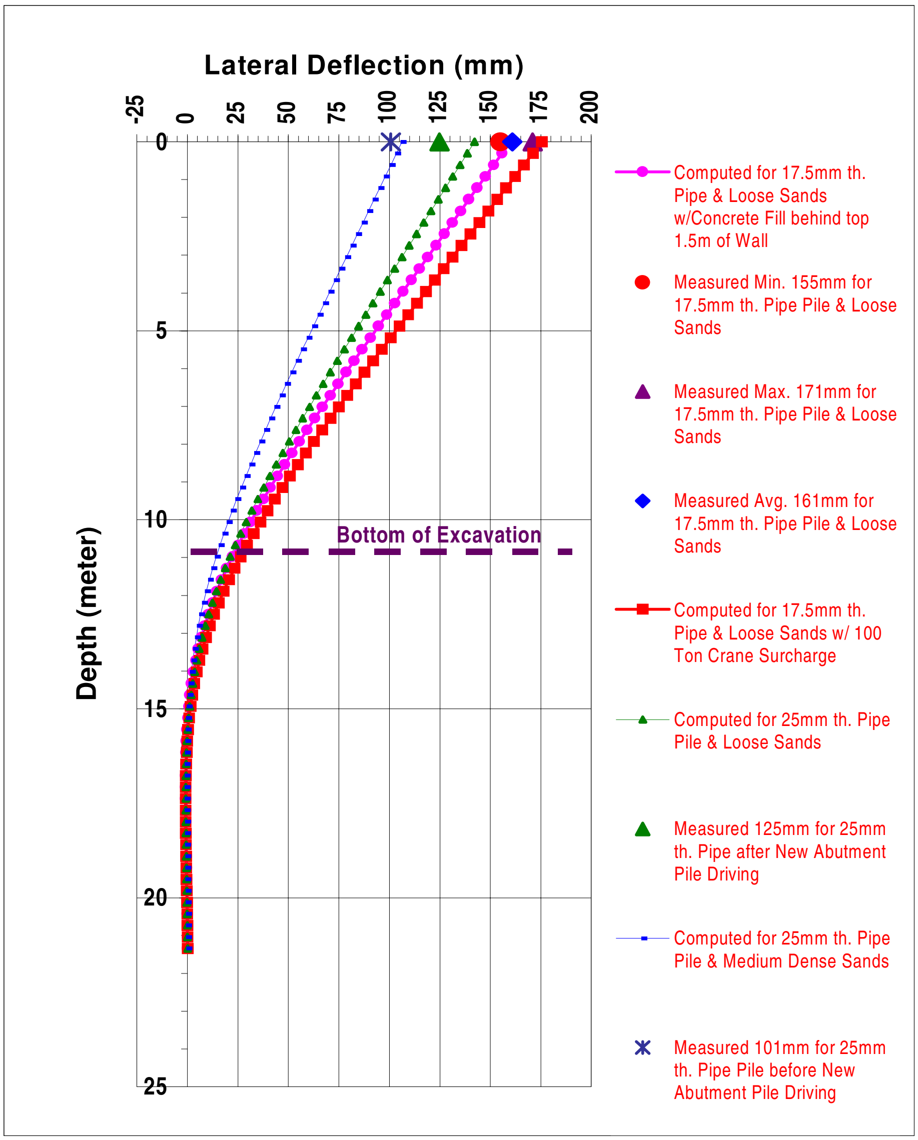

Fig. 4.1. – Pile Lateral Deflection Diagram for the 10.7 m Cantilever Excavation

Figures 4.2 and 4.3 below present the computed pile moment and mobilized soil reaction diagrams, respectively.

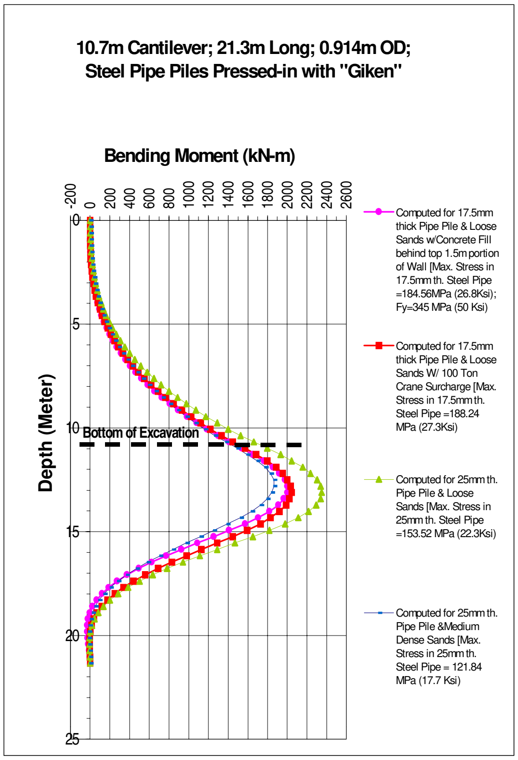

Fig. 4.2. – *Computed Pile Moment Diagram*

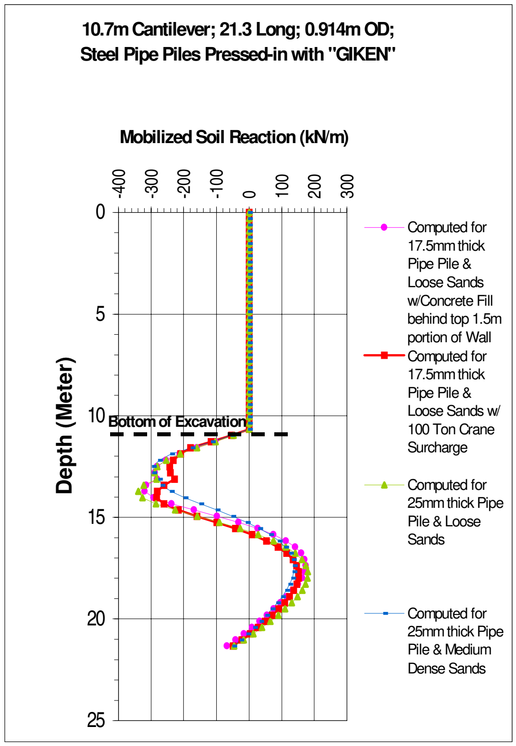

Fig. 4.3. – Mobilized Soil Reaction below Excavation

Photo 4.3 below shows an impressive view of the completed retaining wall in full cantilever at 10.7 m (35 foot) height, with average pile embedment of 10.7 m.

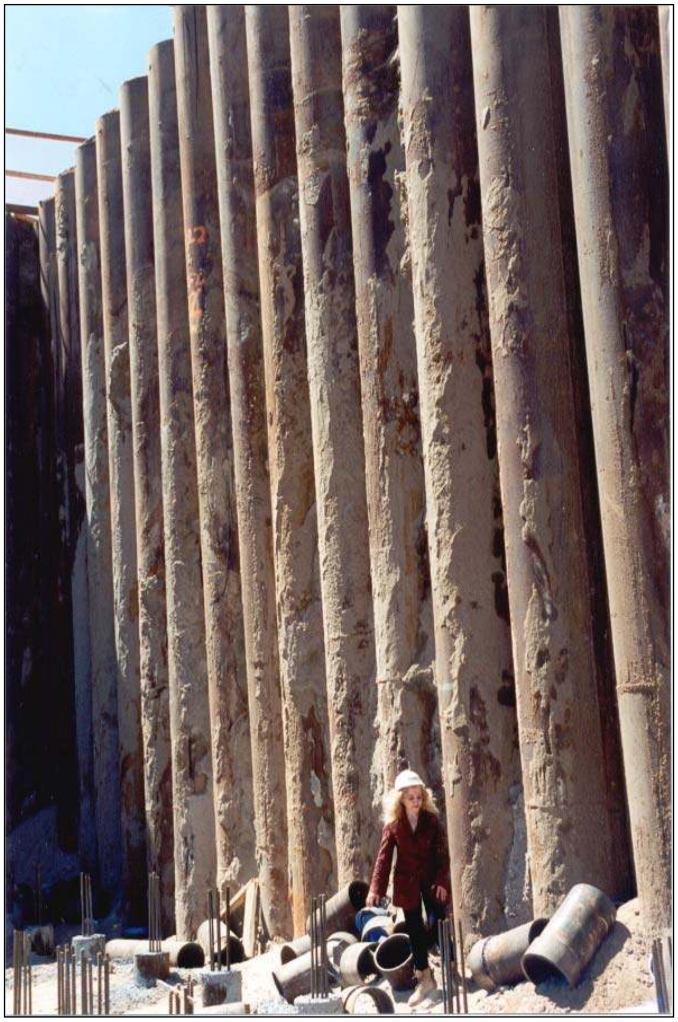

Photo 4.4. – 10.7m Full-Cantilever Support with "Giken" Pressed-in Tubular Steel Piles (note the driven and concreted pipe piles at the bottom of the cantilever wall)

Conclusion

Sound new technology and advanced geotechnical engineering design (considering soil/pile interaction) made this case history a true value engineering change.

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