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ABSTRACT

Construction of high-rise towers in New York City continues to provide exciting challenges for design and construction teams. Sites are becoming increasingly more difficult to build on as “desirable” locations have long since been developed and developers are constructing on sites that were previously over looked. This paper describes two projects that provided unique challenges to the engineers and contractors. The first site is the New York Times Headquarters Tower. This site appeared to be a fairly straightforward foundation design, but became complicated as the subsurface conditions were uncovered. The second case history is the new Bank of America Tower which presented significant design challenges from the outset as it entailed a three basement excavation adjacent to subways and a historic theater façade that required protection. In both cases, close collaboration between the owner, design engineers, construction manager and eventual foundation contractors was required to complete the projects in a timely manner and without adversely affecting adjacent subways, pedestrian traffic, or adjacent historical structures.

INTRODUCTION

The New York Times Headquarters Tower (NYT) and Bank of America Tower (BOA) are the latest towers to be constructed in the ever changing Times Square area of New York City. The sites are in close proximity to the 5 Times Square and 7 Times Square sites that were discussed in Canale et al (2004), as shown on Fig. 1.

The NYT and BOA towers are similar in many respects and different in some. The sites are both large by Manhattan standards, with the NYT site covering 80,000 square feet (SF) and the BOA site covering 95,000 SF. The NYT tower occupies a footprint of 24,500 SF and is 52 stories, while the BOA tower has a footprint of 32,500 SF and is 54 stories. The rest of the sites are developed above grade with 4 to 8 story podium structures that are integral with the towers.

A major difference between the two sites is that the NYT tower has only one basement level, extending about 15 feet below grade, while the BOA building has three basement levels extending 55 feet below grade. Both towers have been critically acclaimed, with the NYT tower winning the New York Construction Project of the Year award for 2007. Currently the NYT tower is completed and occupied while the BOA tower is still undergoing interior fit-out work.

The subsurface conditions at the NYT site contributed to the complexity of the foundation design and construction while the adjacent structures and depth of excavation created design challenges at the BOA site.

General Geologic Setting

The Times Square area is on the Manhattan Ridge, a part of the Manhattan Prong, a formation of old and durable metamorphosed and folded bedrock. Now termed Hartland, this formation was earlier known as the Manhattan or Manhattan Schist Formation. The bedrock has a relatively thin soil cover and an uneven surface. The natural bedrock surface is overlain with a thin mantle of decomposed and/or weathered rock. Overburden soils include glacial and post-glacial deposits and recent fills.

Prior to early development, the area that is now midtown Manhattan consisted of low hills and meadowlands dissected by occasional streams. A stream existed along the west portion of the BOA site. These features are shown on the 1874 Viele survey in Figure 2. A topographic high point is roughly centered around Times Square. It is likely that the original bedrock surface was near the ground surface in the vicinity of the sites as the 1874 map indicates sporadic rock outcrops. The bedrock surface has been altered by construction of buildings and subways for the past 150 years.

Fig. 1. Location of the two projects.

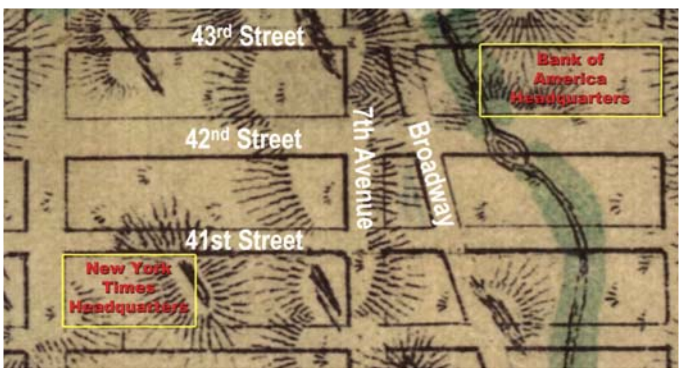

Fig. 2. Topographical Map: Viele, 1874

NEW YORK TIMES TOWER

The NYT Headquarters was designed by Renzo Piano Architects and Fox and Fowle Architects P.C. The project was developed by a partnership formed by Forest City Ratner Companies and The New York Times Company. It is an iconic 52-story tower of 784 feet tall.

Site History

The New York Times site is on the western half of the block bounded by 8th Avenue to the west and 41st Street to the north, as shown in Figure 1. The site is bordered by two New York City Transit (NYCT) subway structures to the north and west and existing structures on the east property line. Sidewalk grades around the site generally slope down towards 8th Avenue from Elev. +49 to +40 (Borough President of Manhattan datum).
Historic atlases and land books of Manhattan dating back to 1899 were researched to identify the former structures. Prior to current development including row housing, a public school, lofts and finally a parking garage on the east side of the site and various height structures to the west.

NYCT structures exist below 8th Avenue and 41st Street, as shown in Figure 1. The subway below 8th Avenue is a box that has a Base of Rail Elevation (BOR) at about Elev. +8.7. The structure is about eight feet west of the property line and was constructed using cut-and-cover methods. An existing active stairway abuts the property line on 40th Street. A similar stairway exists on 41st Street that has been abandoned and paved over.

The Flushing Local subway line beneath West 41st Street borders the site to the north and was constructed in the mid 1920s as an extension of the Queensboro Cross-Town Subway from Grand Central Terminal/Park Avenue to Times Square. Adjacent to the site, the subway was bored through the rock. The BOR in the subway box adjacent to the site is at about Elev.-4.

A pedestrian passageway is below 41st Street, with a base slab that is at about Elev. +29. This passageway was constructed using cut-and-cover methods after the subway tunnel below was constructed.

The NYT tower occupies the western portion of the site and a 4-story podium occupies the balance of the site. One basement level was constructed over the entire site. Foundations for the structure were initially anticipated to be spread footings founded on intact New York City Class 2-65 (as per the local Building Code) rock. However, a series of subsurface investigations showed that a spread bearing solution was not going to be suitable for portions of the tower. This case history reinforces the need for detailed site specific subsurface investigations to avoid costly change orders, even when adjacent subsurface conditions are relatively known.

Subsurface Investigation and Conditions

Mueser Rutledge Consulting Engineers (MRCE) was recently involved in several projects in the immediate vicinity of the site (Figure 1). All investigations at those sites encountered competent bedrock at relatively shallow depths, similar conditions were expected at the New York Times site.

Due to site accessibility issues, it was not possible to make a comprehensive boring program early in the design phase. Instead, a preliminary boring program of six borings was made with truck mounted equipment. Five of the borings were made through the surrounding sidewalk and only one boring was made within the site. None of the borings was within the tower footprint. The borings generally confirmed that competent rock was relatively shallow and that spread footings bearing on rock with an intensity of 20 to 40 tons per square foot could be assumed. The borings extended thirty feet into rock and confirmed a rock quality generally increasing with depth.

As the site became more accessible, a Phase 2 boring program of six additional borings and three test pits was implemented. The 6 borings were made within the site, with only one boring within the tower footprint. Although the borings generally confirmed the results found in the preliminary investigation, the single boring within the footprint indicated a zone of poor quality rock, not encountered in the other borings.

Based partly on the one boring that showed poor quality and lack of borings within the proposed footprint, a Phase 3 boring investigation consisting of three borings was made with a diesel powered skid rig inside existing buildings that were undergoing demolition.. The three borings took approximately 3.5 weeks to complete. Two of the three borings encountered poor quality rock such as highly weathered and decomposed rock to depths of 70 feet below grade. The results were in marked contrast to the previous findings.

Design Recommendations

Upon completion of Phase 3, it was clear that, if conventional footings were to be used, excavations would become unmanageable as the depth to which good quality rock varied across the site and in some cases were 70 feet below grade. Therefore, MRCE recommended three foundation alternatives be considered.

The first alternative was to design a reinforced concrete mat slab to support the tower at a reduced bearing capacity of 8 tons per square foot (tsf). This would eliminate the need for field judgments as to the quality of rock and reduce the uncertainty of the depth of excavation for the bid for the foundations.

The second was to support all of the tower columns on rock-socketed drilled caissons installed through the poorer quality rock and into intermediate or better quality rock. Conventional NYC caissons with capacities of 1,000 to 2,000 tons could be obtained using diameters between 24 and 30 inches. Higher capacities can be obtained using high strength steel in the core.

The third alternative was to retain the current design for spread footings and include a unit price in the bid for installing rock socketed caissons where needed. It is important to note that, at this time, the extent of poor quality rock was not known.

In addition, MRCE recommended that, after the site was cleared, at least one boring be made at each tower column to verify the subsurface conditions. At that point, bids were being solicited from foundation contractors based on the conventional spread footing design. Time pressure made it impossible to evaluate other alternatives.

High column loads of 6,000 to 22,500 kips and relatively wide column spacing, made it difficult to distribute the loads evenly on the mat. The size of the mat made this alternative costly and the solution was rejected. After discussions with the design team, potential contractors, owner, and construction manager Alternative 3 was selected.

Seven additional borings were made with a skid rig prior to demolition of the existing structures. These borings also encountered the soft seams of rock. MRCE was able to determine that the seams were limited, as intact rock was found east and west of the seams. Based on the last investigation, MRCE provided Figure 3 to indicate the column locations affected by the soft rock, those that could bear on converted footings on intact rock and those that were still relatively unknown. Figure 3, along with geologic sections

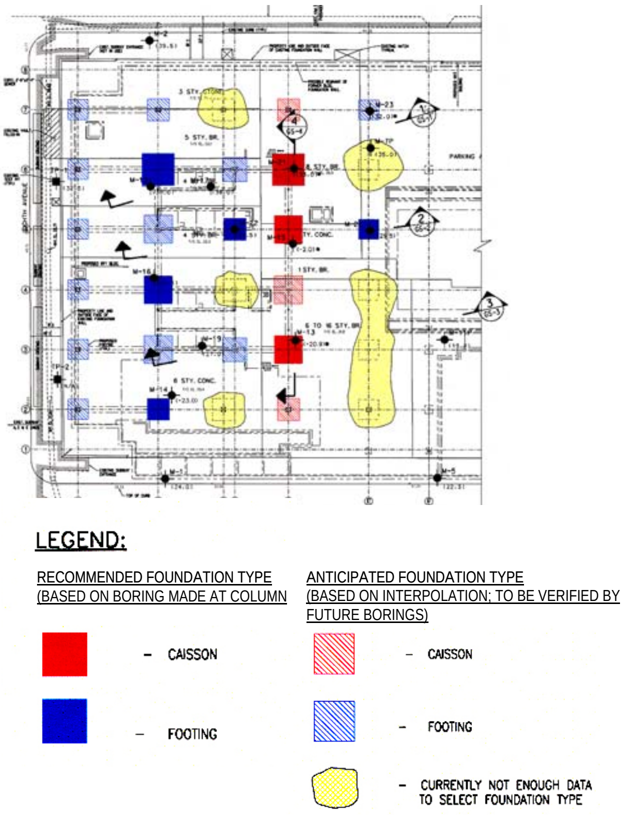

Fig. 3. Preliminary Foundation Recommendations

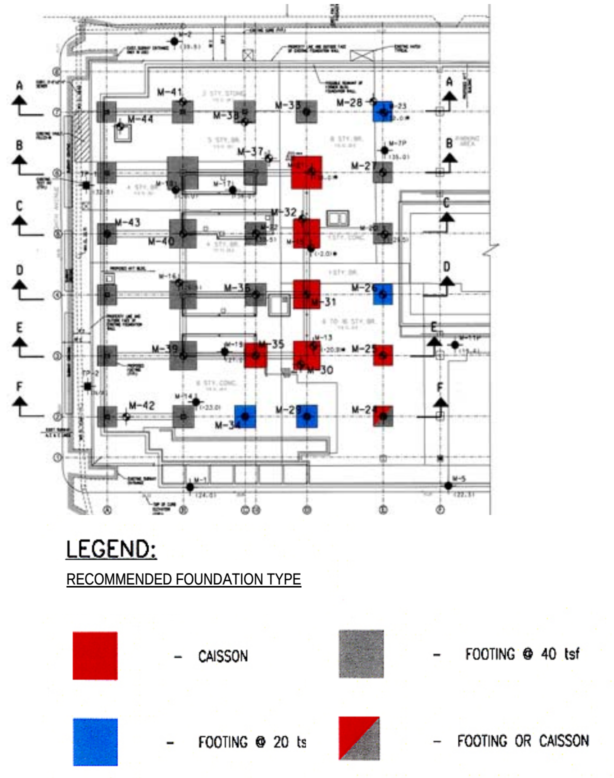

Fig. 5. NYT Final Foundation Design Recommendations

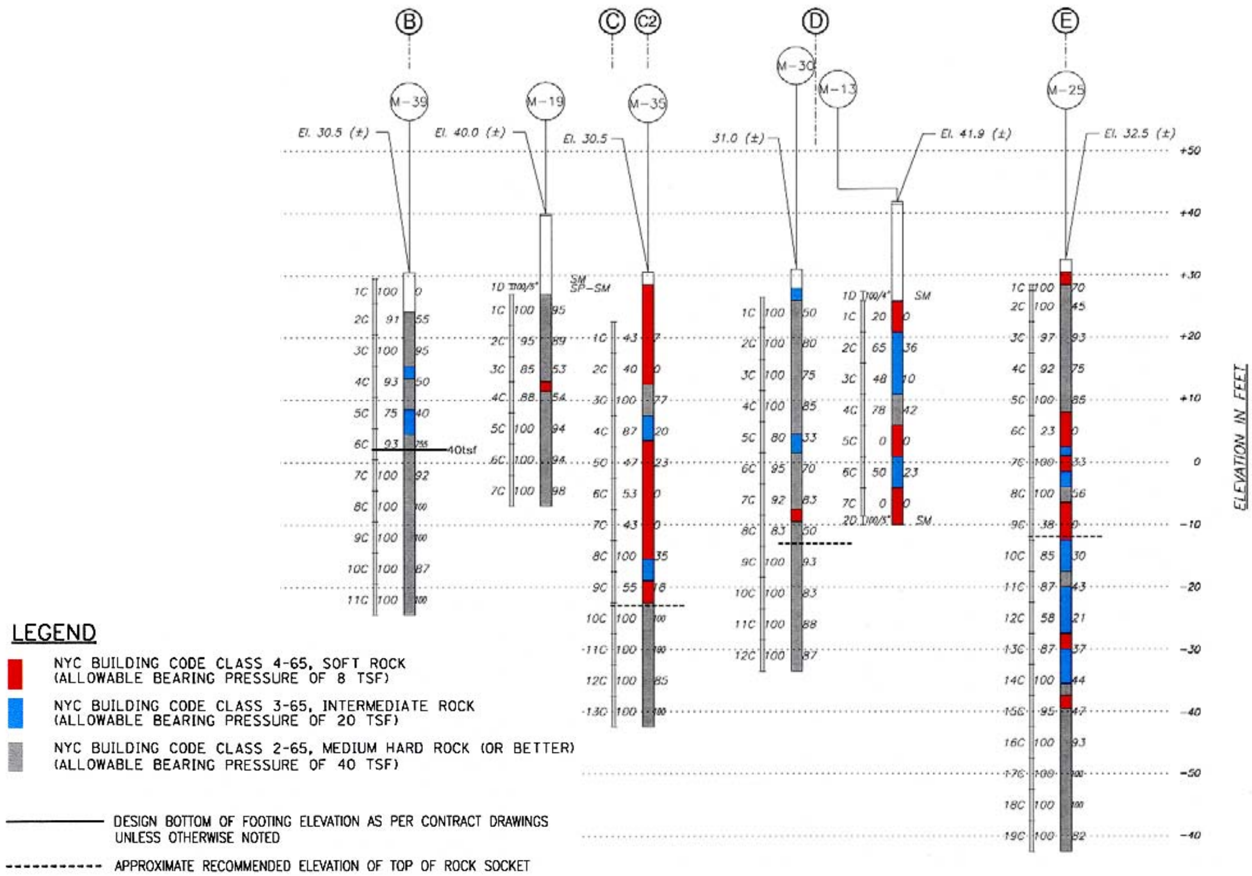

Fig. 4. Typical Rock Quality at NYT

were used by the foundation bidders to estimate caisson numbers and lengths. A typical geologic section is shown in Figure 4. Since the seams of soft rock were discovered in the design phase, the owner was able to obtain competitive bids for the foundation change; whereas if these seams were uncovered during construction, delays and costly change-orders would have resulted.

The foundation contract was awarded with unit prices for lineal feet of caisson installed. Finally when all of the buildings were demolished and removed, 20 additional borings were made such that each column had at least one boring. MRCE prepared a final plan showing the recommended foundation system for each column (Figure 5).

Foundation Construction

Civetta Cousins Joint Venture (CCJV) was the successful bidder for the project and started construction in September 2004 and the foundations were completed in July 2005. CCJV presented a caisson design for review by the design team that consisted of 42 22-inch diameter caissons with allowable vertical capacities between 850 and 1200 tons. The caissons were reinforced with high strength steel bars and installed using a down-the-hole hammer (pictured in Figure 6) to excavate the rock and seat the casings. Grout with a compressive strength of 6,000 psi was used to fill the caissons, as pictured in Figure 7.

Fig. 6. Caisson Rig at NYT Tower

Fig. 7. Grouting of Caissons at NYT Tower

The unique core section consisting of up to 15 #20 (2.5 inch diameter) deformed bars allowed the contractor to adjust the core section to the length of the caisson right after the caisson was drilled. This saved both time and money over a conventional rolled section that would have had to be pre-ordered. The caissons were also designed to resist tension loads on the order of 350 to 675 kips per caisson, limiting the number of rock tie-down anchors.

An aerial photograph during the foundation construction (Figure 8) illustrates the locations of the drilled in caissons. As shown, the limits of the caissons were as MRCE recommended in Figure 5 after all of the borings were completed.

Fig. 8. Overhead View of Caisson Caps at NYT Tower

The rock socket lengths ranged between 13 and 20 feet, with the overall caisson lengths ranging from 31 to 89 feet, with deeper caissons where the deepest soft rock was encountered. Each socket was inspected with a video camera to verify the rock quality in the socket and the condition of the seal to the rock interface.

During construction it was imperative to seat the casings below the zone of soft rock so that a positive seal could be made and the socket cleaned out. MRCE's resident engineer and the foundation contractor worked hand-in-hand to determine the socket depths based on the borings and completed caissons.

The vast majority of the caissons were constructed without incident, but a handful of the 42 had to be grouted and re-drilled in order to achieve a proper seal. As this process is costly and time-consuming, close attention was paid to where the casing was stopped. The balance of the foundations consisted of spread footings bearing on intact rock with bearing capacities between 20 and 40 tsf.

Closing

Although the site was within an area of relatively known subsurface conditions, seams of soft and weathered rock were detected towards the end of the design phase. This finding necessitated that bid documents would provide competitive bids and flexibility to adjust the design based on future borings.

Close coordination between the Design Team, Owner, Construction Manager and Foundation Contractor was essential in providing a foundation system that was cost effective and capable of supporting the tower shown completed in Figure 9.

Fig. 9. New York Times Tower

BANK OF AMERICA TOWER

Bank of America (BOA) Tower is the latest high-rise addition to the Times Square area and is sited on two-thirds of a city block bounded by 4 Times Square to the west, 6th Avenue to the east, and 43rd and 42nd Streets to the north and south respectively as shown in Figure 10. The project consists of a 54-story, 945 feet tall commercial tower with an 8-story podium structure covering the western portion of the site. Three basement levels extend 55 feet below grade over the entire site footprint. Street grade is approximately at El. +62 (Borough President of Manhattan Datum), with the new basement reaching to El. +7.

The depth of the excavation presented a challenge in two respects. One was the support of excavation along 6th Avenue where a deep cut-and-cover subway tunnel and a deep mined subway tunnel existed in relatively close proximity to the site. The second significant challenge was temporarily bracing and shoring the Landmark Henry Miller Theatre façade that was to be re-used in a new theater being constructed as part of the project. This paper will focus on these two aspects of the project.

Site History

The 1885 Atlas indicates that the site was once occupied by row houses with backyards. Eventually, the row houses were replaced with larger commercial structures. By 1916, the 4-story Henry W. Miller Theatre and the 12-story Elks Club had been constructed along 43rd Street. The 20-story Remington building stood along 42nd Street, adjacent to the Elks Club. The 42nd Street Shuttle subway borders the site to the south, about 20 feet from the property line. The base of rail elevation slopes up to the west from about Elev. +33 to +38, or roughly 22 feet below grade in the site vicinity. The shuttle was the first subway constructed in NYC around the turn of the century using cut-and-cover techniques and was opened in 1904.

The B, D, F, and V subway lines, constructed between 1936 and 1939, run beneath 6th Avenue,. The construction was complicated by the presence of the 42nd Street shuttle and variable ground conditions. The base of the existing shuttle tunnel was eventually altered to become the roof of the 6th Avenue subway. However, the subway line closest to the site enters a rock tunnel about 70 feet south of the 43rd Street property line. The tunnel wall is about three feet east of the property line and its base slopes up to the south from Elev. +14 to +19. There are subway entrances at the corner of 42nd Street and 6th Avenue, and at mid-block on 6th Avenue. The corner entrance was constructed in 1938 and extends to within one foot of the southern building line. The entrance on 6th Avenue extends about 3.5 feet from the building line. Both entrances were reconstructed as part of this work.

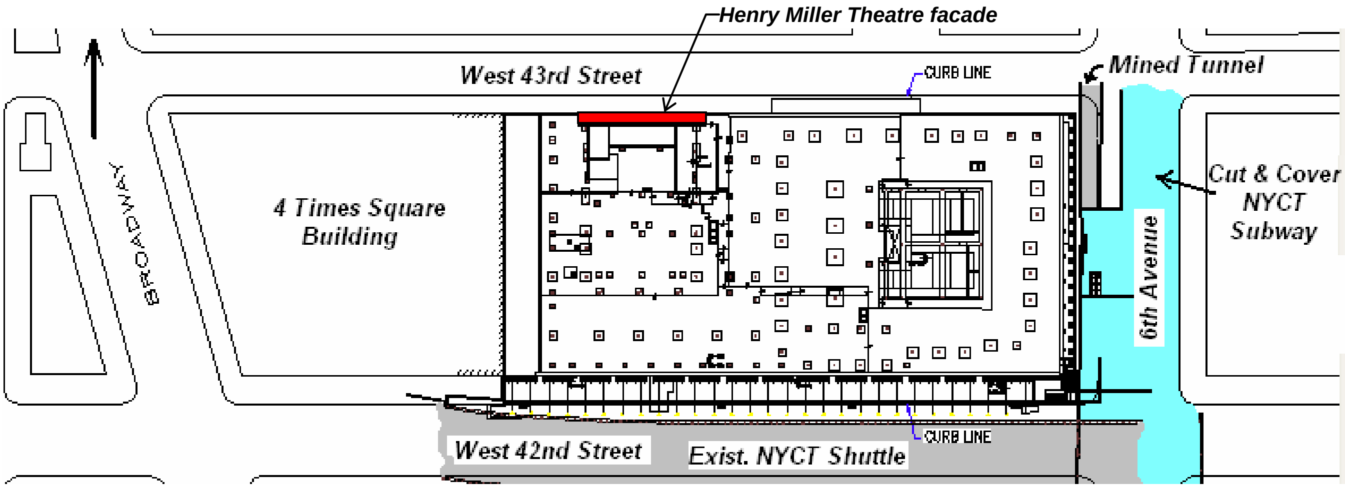

Fig. 10. Bank of America Tower Site Plan

<u>Subsurface Investigation and Conditions</u>

MRCE performed a subsurface investigation in two phases as access to the majority of the site was not available during the early stages of the design process. Five preliminary borings were performed in June of 2003 and 16 borings were made for the final investigation in February 2004.

Three of the borings were drilled through the sidewalk, with truck-mounted rigs to determine overburden and rock characteristics at critical locations adjacent to existing subway lines. Two borings were drilled from within existing structures on the site, with an electric-powered skid rig. A piezometer was installed in one boring to monitor groundwater levels.

Twelve of the borings were made from within existing structures through the sidewalk. Groundwater levels were monitored by two additional piezometers. Two borings were drilled on 6th Avenue to determine the rock depth and quality above the mined subway tunnel. These borings were limited to approximately 5 ft above the tunnel, a total depth of 25 feet.

Soil samples from Standard Penetration Tests (SPTs) were obtained through the overburden soil and bedrock cores were obtained. To determine the strike and dip angle of joints within the bedrock units, one boring was made with an oriented core barrel containing scribes that mark the core in advance of extraction from the ground. This permits the evaluation of rock joint orientation effects on excavation and, hence, the impact on nearby structures.

The subsurface conditions varied significantly across the site. A relatively deep rock profile, extending 50 feet below grade, was identified at the western portion, in the vicinity of the historic stream bed. This rock depression was filled with decomposed rock, glacial till, alluvial sands, and silts.

In the vicinity of the 6th Avenue tunnel, rock was found at 10 to 20 feet below sidewalk grade, overlain by a man-made fill. A thin layer of decomposed rock was encountered in some of the borings.

The rock consisted of generally hard gneissic schist to schistose gneiss with occasional zones of intermediate quality rock. An intrusion of serpentine/amphibolite rock was encountered in the boring on the corner of 43rd Street and 6th Avenue. Rock recoveries in the vicinity of 6th Avenue were generally good and Rock Quality Designations (RQD) varied from 12% to 100%, with an average of 62%. The rock quality generally increased with depth. A typical geologic section showing the boring results and the relationship of the subway tunnels with the location of the subway is shown in Figure 11.

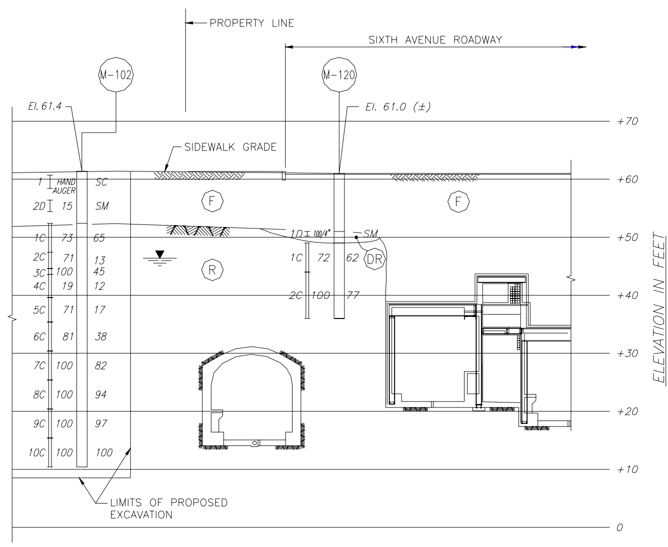

Fig. 11. Typical Geologic Section at BOA along 6th Avenue

Design Recommendations

As the basements were 55 feet deep, MRCE recommended that the new columns be supported on conventional spread footings, with allowable bearing capacity of 40 tons per square foot (tsf). Uplift forces due to wind loads were resisted by permanent pre-stressed double corrosion protected rock tie-down anchors.

To relieve water pressures between the basement slab and rock surface, MRCE recommended an underdrain system leading to multiple sump pits was recommended.

The most significant challenges of this project and the process of excavating the site and protecting adjacent structures is described in the following sections.

Excavation Support along 6th Avenue

The new structure is set back from the east property line. Therefore, the old foundation wall of the existing building was used to retain soil above the rock surface. Stability of the existing concrete foundation wall was provided by a series of 4 ft x 4 ft concrete pillars at 10 ft spacing resting on rock (Figs. 4, 6, & 7). Every concrete pillar is restrained by a prestressed tiedown anchor. Rock stability during excavation was analyzed using classical wedge analyses concepts. Since rock joints dipped at 60 to 70 degrees mostly towards the excavation as indicated by the oriented rock core data, two-dimensional analyses were used. Two design sections were considered: a) at the cut-and-cover tunnel (Fig. 12), and b) at the mined tunnel (Fig. 13).

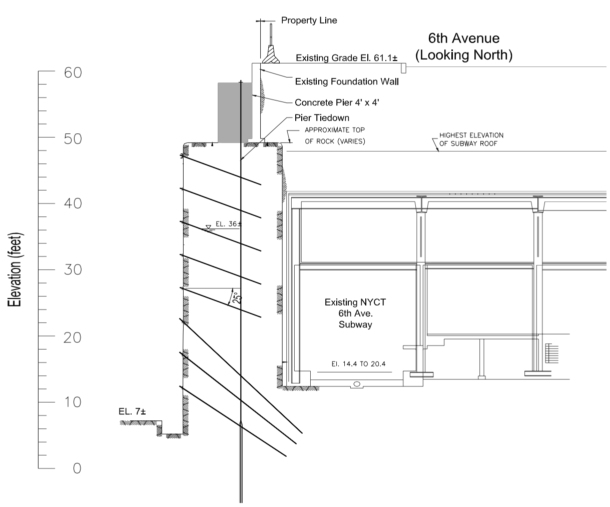

Fig. 12. BOA Section at 6th Avenue along NYCT Cut & Cover Tunnel

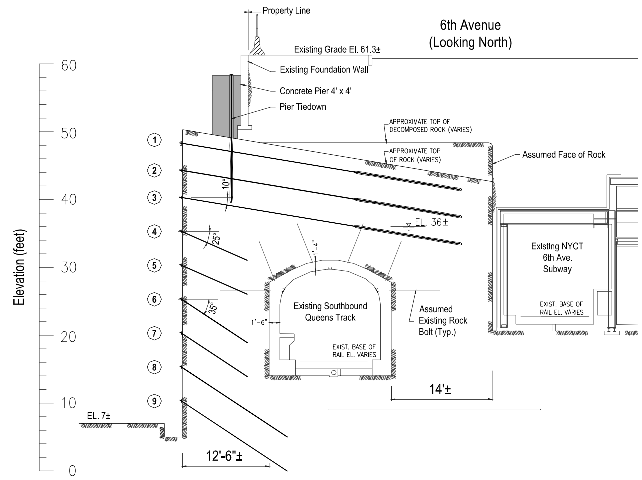

Fig. 13. BOA Section at 6th Avenue along NYCT Mined Tunnel

At the cut-and-cover tunnel, rock wedge stability was evaluated for all stages of excavation. Initial contract drawings envisioned using a system of three levels of rakers resisting sliding forces from the 12 ft wide rock pillar west of the cut-and-cover tunnel. Short rock bolts were to be used to prevent localized block failures, as preferred by the Contractor to simplify construction. Rock bolts were designed to resist driving forces from soil, water pressure, surcharge, and from the cut-and-cover tunnel. The upper bolts were 10 ft long 1-1/4" O.D. grade 150. The lowest bolts were 16 to 22 ft long 1-7/8" O.D. grade 150 threaded bars; as seen in Figure 12, the lowest three levels of bolts were inclined at steeper angles in order to avoid the subway tunnel.

Supporting the excavation along the mined subway tunnel was conceptually more challenging than along the cut-and-cover section. Removing rock west of the tunnel would compromise confinement of the rock arch and tensile stresses in the tunnel roof could potentially develop. The mined tunnel was likely constructed after the cut-and-cover tunnel. The close proximity between the two tunnels probably had some effect on the arching behavior of the mined tunnel.

Available construction drawings from the 1930s did not show any reinforcement in the existing horse-shoe shaped concrete liner. As a result, increases in tensile stresses on the liner could result in structural damage. Thus, in order to avoid stress increases on the tunnel, the existing rock arch action had to be preserved. This was achieved by providing a series of prestressed rock anchors and passive bolts. The top three anchor bolt levels above the mined tunnel were designed to create a prestressed rock-beam that rests on two supporting rock pillars (Figure 14).

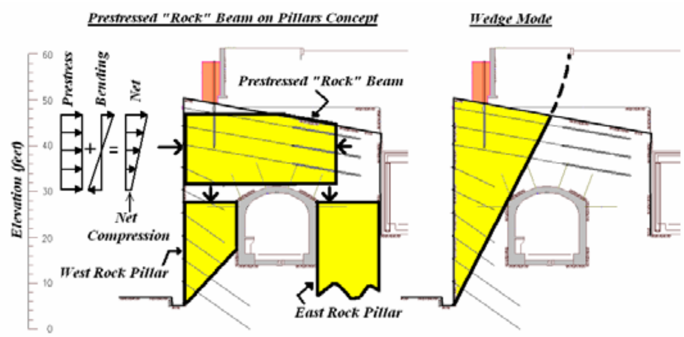

Fig. 14. Conceptual Analyses along NYCT Mined Tunnel

These top anchors were actively loaded to maximize the likelihood that rock over the mined tunnel would stay under compression and not experience any tensile stresses.

Bearing stresses induced by the "rock beam" on the rock pillar were evaluated and found to be within New York City Building Code presumptive values (20 tsf). In order to provide rock-pillar integrity, four levels of passive rock bolts were installed (Levels 4, 5, 6, and 7). These rock bolts effectively "stitched" the rock pillar together thereby limiting joint movement under the weight of the "rock beam" above.

The rock bolts were 10 ft long 1-1/4 inch O.D. threaded bars, grade 150 (same as used along the cut-and-cover tunnel), generally installed in a 5 ft grid. A series of wedge stability analyses were also performed for critical stages of the excavation. Rock cohesion was ignored, and friction along joints was used. The upper and lowest levels of anchors are basically intended to restrain "full" wedge failure modes. In contrast to conventional wedge analysis methodology, safety factors were not defined on the anchor capacity (i.e. service design). Instead, safety factors were evaluated based on available shear strength vs. mobilized shear strength ratio on examined joint conditions. This safety factor definition is consistent with finite element method safety factor definitions. Safety factors of 1.5 or more were targeted to minimize the potential for joint movement and damage to the mined tunnel.

Similar modes of potential failure (i.e. wedge failure) were also evaluated with finite element models. The analyses indicated insignificant lateral movements of the tunnel and validated the concept of conventional prestressed beam resting on two rock-pillars. The upper section of Figure 15 shows a preliminary finite element model. Crosses indicate major and minor principal stress directions. The subgrade in this preliminary model was at El. +0, or 7 ft deeper than the as-built subgrade as the design was not finalized at the time. In preliminary models higher stress concentrations were observed at assumed rock joints that modified arching stresses. Subsequently after construction was completed, a more detailed finite element model has been performed reflecting as-built conditions with observed rock jointing patterns in order to match the measured behavior is shown in Figure 16. The benchmarking finite element model can be seen in the lower section of Figure 15.

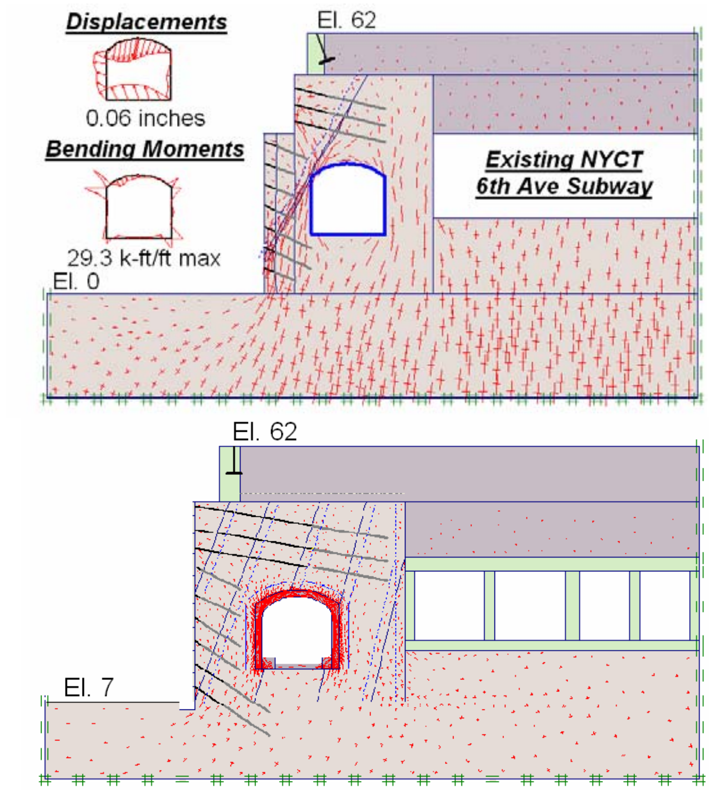

Fig. 15. Preliminary & Benchmarked F.E. Model

Subway Monitoring

Monitoring of the subway structure utilized seismographs to measure vibrations and strain gauges to measure changes in strain in the subway tunnel. Wang Engineering Services installed fifteen strain gauges along the length of the tunnel in arrays of three, placed on the concrete walls and crown of the tunnel liner. The monitors were connected to remote sensing devices. Results of the strain gauge monitoring program are shown in Figure 16 as changes from stress levels at the time of building construction. The crown experienced a decrease in confining stress ranging from 150 psi to 300 psi when final subgrade was reached. Tunnel walls experienced similar increases in compression. Most of the changes in stress levels were observed after excavation progressed beneath the third anchor level. The finite element benchmarking was successful in replicating most of the observed stress behavior during excavation. A photograph of the completed rock cut is shown in Figure 17.

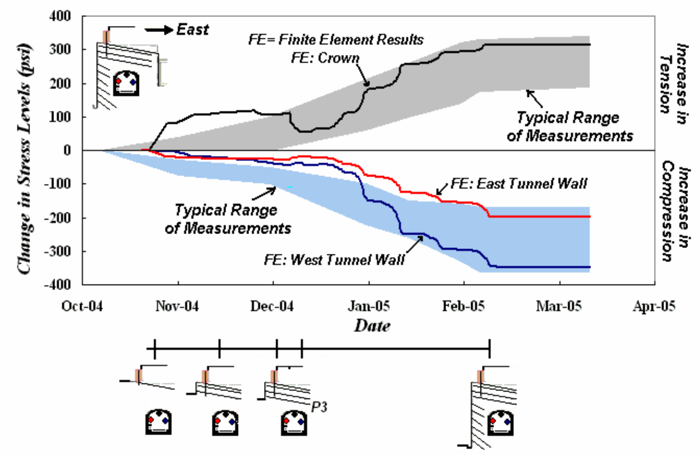

Fig. 16. Typical Range of Measured Strain Gauge Data vs. P.E. Model

Fig. 17. 6th Avenue Rock Cut

Henry Miller Theater Façade Preservation

The Henry Miller Theater was constructed in the early 1900s and was considered a historic structure. The developer had planned to build a new theater in the same location, therefore an effort was made to preserve as much of the old theater as possible to incorporate into the new one. The major preservation piece was to save the façade in place and incorporate it into the new theater. The Empire State Development Company (ESDC), who was in charge of the preservation of the façade, would not let the developer tear down the façade and rebuild it; therefore, it had to be preserved in place.

The façade was primarily constructed with brick with ornamental stone surrounding windows and doors. It is 86 feet long, 50 feet tall and about 4 feet thick. Complicating preservation was the fact that the excavation would extend some 30 feet below the base of the façade and rock jointing was unfavorable along this face of the exaction.

Three test pits and three borings were made in the vicinity of the theater façade to determine the subsurface conditions. The borings indicated that rock was between 19 and 21 feet below grade and was of varying quality. The test pits indicated that the foundation for the façade extended to either weathered rock or intact rock. The foundation wall for the façade consisted of several brick courses and was about two feet thick.

MRCE Recommendations

Based on the boring and test pit results, the following recommendations, summarized in Figure 18, were made for the design and installation of temporary support for the theater façade:

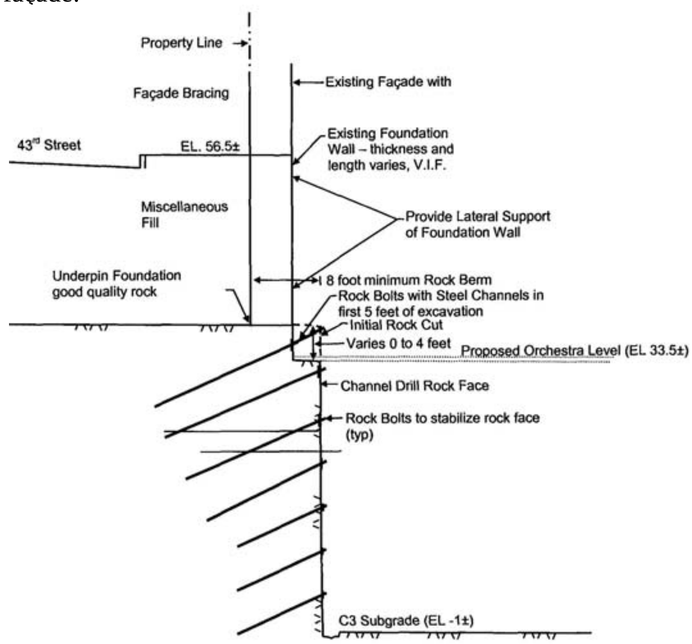

Fig. 18. Conceptual Section Adjacent to Henry Miller Theater

1. Laterally brace the above grade portion of the façade from the sidewalk side, supported on micro-piles socketed into rock. The micro-piles were designed to resist the compressive and tensile forces from the support truss.

2. Once the lateral bracing is installed, detach the façade and demolish the balance of the theater. As the excavation progresses, lateral bracing of the below grade foundation wall would be required. This was achieved with tiebacks installed to rock.

3. Once excavation reached the base of the foundation wall, the wall was to be underpinned to good quality rock.

4. Upon completion of underpinning, channel drilling of the rock was required in advance of further excavation to

limit vibrations on the façade. As rock jointing was unfavorable along this face, we recommended that the excavation be stepped out so that there were four feet of rock in front of the inside face of the façade. This limited the concern of loss of small rock block failures undermining the façade. In addition, pattern rock bolting to support the rock face was recommended.

5. A real-time monitoring system consisting of seismographs and tiltbeam sensors were recommended to monitor the structure during construction.

Construction

MRCE's recommendations were used by the contractor, Civetta-Cousins Joint Venture (CCJV) to create shop drawings that provided details of all elements to shore the façade and create the basement excavation. These shop drawings were reviewed by the design team as well as the ESDC.

Fig. 19. Above Grade Theatre Façade Stabilization

The above grade theater façade stabilization structure is shown in Figure 19. Figure 20 shows the final cut with the existing walls, underpinning required and subsequent pattern bolting. Multiple levels of underpinning were required at the west and east end of the façade. This work was done with negligible recorded settlement or tilt of the façade and with vibrations generally below the criteria of 0.62 inches per second (ips). Façade preservation was a mini-project within the overall project as bi-weekly meetings were held with the ESDC, the design team and contractor to review the monitoring results, current work and future work. This close coordination allowed the work around the Henry Miller Theater Façade to proceed without any significant delays or damage to the façade.

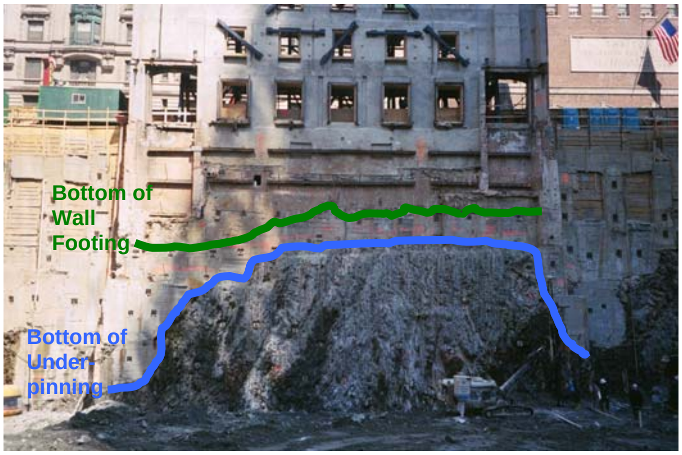

Fig. 20. View Looking North at Henry Miller Theatre

CLOSING

Creating deep basements in an urban environment has always been a challenge. Moreover, underground construction in increasingly congested environments is especially challenging as demand for prime real estate space intensifies. The Bank of America tower project, shown nearly completed in February 2008 in Figure 21, is an excellent example of what is likely to follow in the new century as excavations are constructed next to, and below subways, utilities, and basements in scenarios likely not anticipated by the original designers.

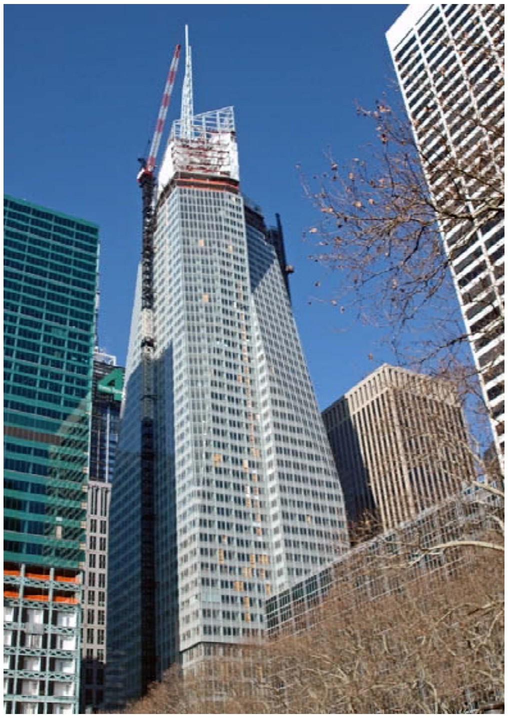

Fig. 21. The Bank of America Tower

Underground design and construction in complex site conditions requires thorough evaluation. Finite element analyses were able to replicate measured field behavior of the underground structures. Complimenting conventional analyses and advanced finite element methods yields a balanced and comprehensive design approach that can be particularly insightful.

Close coordination between owners, engineers, construction managers and contractors is expected in completing foundation work in complicated urban sites.

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