Table 1. Secant Pile Design Parameters

Excavation Heights	Soil At-Rest Pressures	Building Footing Vertical Surcharge Loads	Secant Pile Parameters	Bracing Levels	Top Tier Anchor Loads (Anchors spaced at 8 feet c/c typ.)
25 to 40 feet (7.6 to 12.2 m)	56 psf per foot of depth in glacial till (8.86 kPa/m)	500 to 12,000 psf (24 to 575 kPa)	30-in. (760 mm) dia. W18x130 core beam 4,000 psi (27.6 MPa) grout Spaced at 4-feet (1.2 m) c/c	2 to 4	95 kips to 160 kips (422 to 712 kN)

Limit equilibrium analysis was used to verify the FEM design results. Lateral bracing loads were calculated to be within about 15% of bracing loads determined using the FEM analysis. The higher of the two values was used in design. The FEM model output is provided as Figures 4a and 4b.

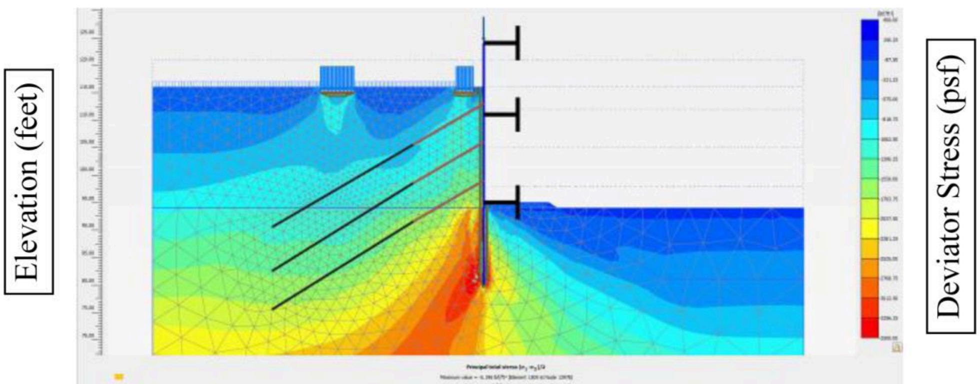

Figure 4a. FEM Section at the Wesley House

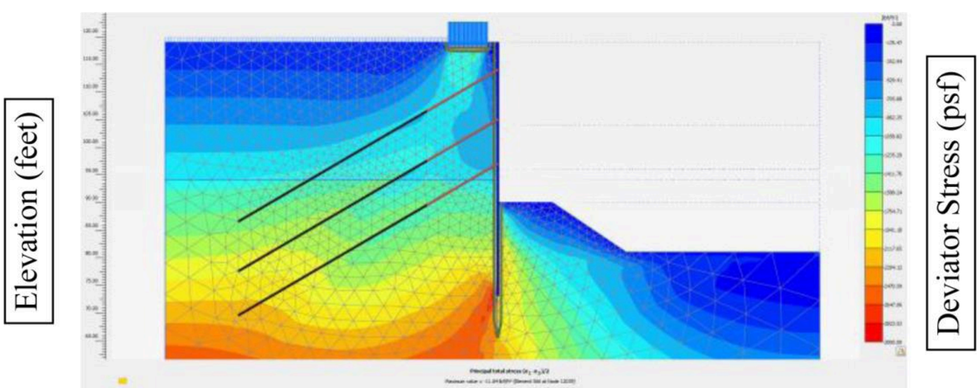

Figure 4b. FEM Section at the western-most row-home in the "U"-shaped site

The final design consisted of 29.5-inch-diameter (750 millimeter) secant piles with a six-inch overlap between adjacent piles to create a continuous secant wall. All secant piles were filled with grout having a minimum compressive strength of 4,000 psi (27.6 MPa) at 28 days. A W18x130 Grade 50 core beam was installed in all secondary secant piles. The secant piles were socketed at least 10-feet (3 m) below subgrade into the glacial till.

Two to four levels of lateral bracing (number of bracing levels increased with increased excavation depth) were provided to limit lateral deflections to less than a ¼-inch. Similar to the soldier pile and lagging system, the primary lateral bracing option was self-drilling hollow bar soil anchors. Anchors with tensile capacities of up to 165-kips (735 kN) were designed to be installed at every other primary secant pile. A continuous double-channel wale was designed to transfer load from the core beams to the anchors. At the corners of the "U," the soil anchor geometry was modeled using three-dimensional software to optimize the anchor spacing and angles to avoid intersecting anchors. An image of the model is provided as Figure 5.

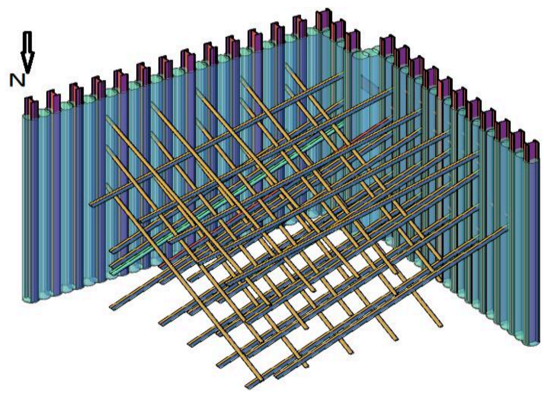

Figure 5. Three-Dimensional Model of Soil Anchors at the Eastern Inside Corner of the "U"-shaped Site.

An owner of one of the row homes did not agree to an access agreement with New York Presbyterian Hospital to allow soil anchors drilled below his property. Loads were spanned to outside of the 25-foot (7.6 m) wide property with four levels of W16 wales. The top three levels of wales were braced by soil anchors with capacities of 180-kips (800 kN) at either ends of the wales. The bottom level wale was braced by two W18 rakers, with axial loads of about 250-kips (1,110 kN) per raker. The rakers were temporarily braced to grade beams for the new building. The secant pile wall detail and a photograph of installation is provided as Figure 6.

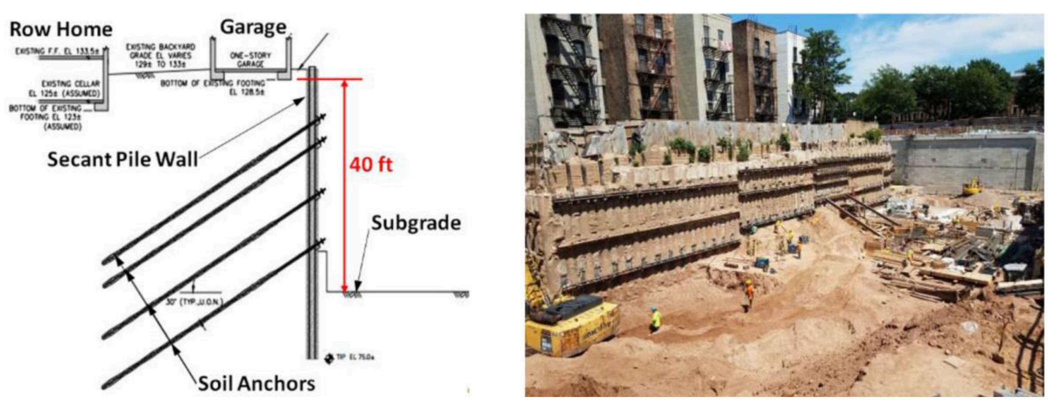

Figure 6. Secant Pile Walls Detail and Photograph

Secant Walls as Foundations

The inside-face of the secant walls was about 4.5-feet (1.4 m) inside of the site lot lines. The secant walls were designed for support of exterior building columns to avoid the need for a new foundation wall to further encroach on proposed program area. The secant piles were designed to support permanent (at-rest) earth pressures and adjacent building pressures when braced by the new cellar and first floor slabs. Exterior column loads would be distributed to the core beams in secondary secant piles by a reinforced-concrete cap beam at the top of the secant pile walls. Each secondary secant pile was designed with an allowable compressive capacity of 75 tons (670 kN).

The project team wanted to limit differential settlements between adjacent spread footings (support for interior columns), the mat foundation (support for the building core), and the secant walls (support for exterior columns) to less than ¾-inch (1.9 cm). Settlements of the new footings and mat foundation bearing in the dense glacial till were estimated to be about ¾-inch (1.9 cm). With a socket depth of 10-feet (3 m) below foundation subgrade, absolute settlement of the secant pile wall was estimated to be about 1 inch (2.5 cm).

SUPPORT OF EXCAVATION CONSTRUCTION

The contractor proposed creative methods of soldier pile, soil anchor, and secant pile installation to decrease the below-grade construction duration, which were used upon review by the project team. Details about the installation methods and successes and challenges of installation are described in the following sections.

Soldier Piles

The 139 soldier piles were installed by advancing a single section of casing with a down-the-hole air-hammer inside of the casing. The contractor proposed using an air hammer to advance the casings through the obstructions and boulders in the historic fill and glacial till. The design team took no exception to this method of installation provided that there were not any signs of sidewalk movement or voids while advancing the casing.

Solider piles were installed to the target depth at a consistent installation rate that met the project schedule. Cracks and voids were not observed in the adjacent sidewalk and street during soldier pile installation and no soldier piles needed to be re-drilled. Soldier pile depth and air pressures were closely monitored during installation. The tip of the hammer remained at the tip of the casing as the casing was advanced. Air flush was observed returning from inside of the casing. As excavation progressed, no voids were observed around the soldier piles, which is indicative that the air from the hammer did not erode soil outside of the casing limits during installation.

Soil Anchors

The contractor proposed to use self-drilling hollow-bar anchors for installation of the 494 anchors to brace the support of excavation. Grout was continuously pumped through the anchor bars and drill-bit under pressures of up to 150-psi (1,030 kPa) as the anchors were drilled to the design length. Each anchor was proof- or performance-tested (10% of the anchors were performance tested) to 133% of the design load in accordance with Post Tensioning Institute standards. Anchors at the soldier piles and lagging system were locked-off at about 70% of the design load. Anchors at the secant piles were locked-off at about 80% of the design load to limit deflection.

Installation of 487 of the 494 anchors (approximately 98.5%) were successfully drilled through historic fill and boulders to the target depth. Three first tier anchors at the southeastern part of the site failed proof or performance tests. These anchors were drilled at the part of the site where soft silts with organics were encountered in the borings. The soft silt was originally expected to be above the bond zone of the tiebacks. Replacement anchors were drilled about 1 foot (0.9 m) from the failed anchors, with bond zones that extended about 10-feet (3 m) longer than the original design. The new anchors passed proof testing.

Four anchors at lower levels of the secant walls could not be installed to the target length by the contractor. Boulders likely damaged the drill-bit, as inferred from difficult drilling through obstructions for over half of the anchor length at the failed anchors. At installation of other anchors throughout the site, difficult drilling was encountered generally less than 30-percent. Replacement anchors were drilled one-foot from the failed anchors to the same anchor lengths and successfully passed proof testing.

Anchors installed for bracing of the secant piles at the inside corners of the "U" were drilled without intersecting each other, which verified the benefit of the three-dimensional design model. Testing of the anchors verified the design allowable bond stress of 12 psi (83 kPa) at all anchors except for anchors drilled through the soft silt. Measured bond stress for anchors below the first tier was over 14 psi (97 kPa).

Secant Piles

The contractor recommended installing the 230 secant piles using either of two methods: the Kelly-bar method and the Cased Continuous Flight Auger (CCFA) method. The contractor believed that these installation methods were necessary to productively advance the secant piles through obstructions to meet the demanding project schedule. Photos of the secant piles installation using the Kelly-bar and CCFA methods are provided as Figures 7a and 7b.

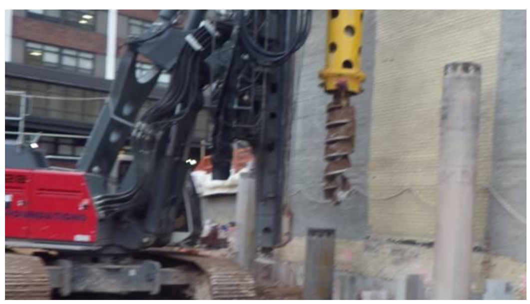

Figure 7a. Secant Pile Installation via the Kelly-bar Method

Figure 7b. Secant Pile Installation via the CCFA Method

The Kelly-bar drilling method was comprised of drilling temporary casing in 9.8-feet (3 m) sections to the final pile depth. An auger was used to remove soil from inside of the casing as each casing section was advanced. The auger was removed upon achieving the design secant pile depth, a core beam was installed (if constructing a secondary secant pile), and the casing was then gravity-filled with concrete as each casing section was removed.

The CCFA drilling method comprised of drilling a single section of temporary casing with a continuous flight hollow-stem auger inside the casing. The auger tip would remain at approximately the same elevation as the casing tip during drilling. Once the design depth was achieved, the auger was raised slightly, and concrete was pumped through the auger-stem and out of the auger tip at the bottom of the temporary casing. The auger and soil that the auger drilled through, along with the casing were then extracted simultaneously as the concrete was pumped through the auger stem to the top of the secant pile. A head of concrete was maintained inside the augers while extracting the casing. A core beam would then be installed inside of the concreted pile if constructing a secondary secant pile.

This is reportedly the second project in New York City in which the CCFA method was used. Two concerns about the CCFA method were identified early in the project: (1) boulders prohibiting the advancement of the auger to the required design depth, (2) concrete becoming clogged in the auger stem during extraction of the casing, and (3) grout setting prior to installation of the core beam.

Both secant pile methods were able to effectively penetrate through boulders and obstructions. Drilling rates decreased when drilling through boulders; however, the contractor successfully drilled every secant pile within the project schedule. No gaps were observed in the face of the exposed secant walls during construction. Boulders exposed during the excavation were up to 10-feet-thick, which verified the difficult drilling conditions at the site.

Grout became clogged in the auger tip in two of the first five CCFA piles attempted. The contractor had to remove the casing before filling the open hole with concrete because the auger and casing couldn't be detached from the drill rig to repair the concrete clog. Monitoring data indicated that the adjacent row home moved laterally about $^{1}/_{8}$-inch (3.2 mm) after the casing was removed. The procedure was almost abandoned, but the contractor re-tooled their drill rig to allow for the casing to be detached upon concern from the project team that this could become a common occurrence. They also made adjustments to the grout mix to improve its pumpability.

CCFA also proved difficult for the first four attempted secondary secant piles, because concrete was setting faster than anticipated. The core beam can't be installed until the auger and casings are removed, and the concrete was setting before the core beam could be fully inserted. For the rest of the project, the contractor used the CCFA method mainly for primary secant piles, because they were able to install primary piles more efficiently with CCFA than with the traditional Kelly-bar drilling method.

Axial Compressive Load Tests on Secant Piles

Production secant piles at the secant pile walls could not be tested because there was not enough area for a pile test frame without encroaching on the adjacent property. Also, the piles would be required to be tested before the secant walls were installed and site-wide excavation could commence. A static axial compressive load test was conducted on two sacrificial test piles installed from grade at the eastern part of the site (identified as Test Pile TP-1A and TP-2A) to establish the allowable axial compressive design capacity of the secant piles, in accordance with ASTM D1143 standard procedures.

The load tests were conducted to verify that the bottom 10-feet of the piles could support the proposed axial loads, because the secant pile walls would be excavated (to subgrade) at the front face of the wall and the support from soils at the back face of the wall (above subgrade) was ignored in the secant pile design. The surface area of a single secant pile was calculated to be similar to the surface area of the front and back face of a secondary secant pile plus half of the two adjacent primary secant piles. Therefore, axial load testing the bottom 10-feet (3 m) of a sacrificial single test pile was determined to provide similar results to load testing a segment of the continuous secant wall (without interference from other secants not being loaded).

Test Piles Installation: Pairs of Arc Weldable 4000 strain gauges were installed along the core beams in the test piles -- about 2 to 3.5-feet (0.6 to 1.1 m) below the pile head, about 10-feet (3 m) above the pile tip, and about 1-foot (0.3 m) above the pile tip – to measure the amount of load being transferred to the bottom 10 feet (3 m) of the pile during testing. Each pile was installed to about 10-feet (3 m) below the proposed general subgrade. Test Pile TP-1A was installed using the Kelly-bar method. A W18x130 core beam with 4,000 psi (27.6 MPa) concrete that matched the secondary secant pile design was installed in the pile. Test Pile TP-2A was installed using the CCFA method. Based on the results of the load test on TP-1A, a friction-reducing jacket was installed on a W14x120 core beam (except for the bottom 10-feet [3 m]) to facilitate test load transfer to the bottom 10-feet (3 m) of the pile. The core beam was a different section from the design core beam to allow for fitting the friction-reducing jacket. Changing the core beam size did not affect the test because the pile interaction with the soil was being tested, not the lateral bending capacity.

Load Test Set-up: A 500-ton (4,450 kN) hydraulic jack was used to apply load on the pile while reacting off of a load testing platform. The platform consisted of a reaction frame constructed of steel beams connected to four reaction piles; each reaction pile was designed and proof-tested for 112.5-tons (1,000 kN) of tension capacity. The top of grout was at a similar elevation as site grade prior to excavation and the core beam extended about six-inches (0.15 m) above grade. A steel base plate was placed on top of the core beam and served as a platform surface for the hydraulic jack and dial gauges. A photo of the load test set-up is provided in Figure 8.

Figure 8. Sacrificial Secant Pile Axial Compressive Load Test Set-Up

Pile movement was measured using three extensometer dial gauges reading to the nearest 0.001-inch (0.025 mm). A mirror-wire-scale assembly reading to the nearest 1-millimeter was used as a secondary method of measurement. The gauges and mirror-wire-scale were mounted on a reference frame independent of the load frame. The measured gross settlement was recorded as the average settlement measured from the three dial gauges at a load increment.

The 150-ton (1,335 kN) test load (200% of the proposed design load) was applied in increments of about 18.75 tons (165 kN), following the “Standard Loading” procedures of ASTM D1143. After 200% of the design load was recorded, additional load was applied in increments of about 20 tons (178 kN) until the strain gauges at 10 feet (3 m) from the pile tip indicated that the 150-ton (1,335 kN) load was transferred to the bottom 10 feet (3 m) of the test pile. The additional applied load was stopped at 450-tons (4,000 kN - 600% of the proposed design load), which was the maximum capacity of the load frame.

Results: Test Pile TP-1A was loaded to the load-frame capacity of 450 tons (4,000 kN). At 450 tons (4,000 kN) of applied load, the pair of strain gauges at 10-feet (3 m) from the pile tip recorded the same strain as recorded by the pair of strain gauges 3.5-ft (1.1 m) from the top of the pile (gauges closest to the applied load) indicating that about 77 tons (685 kN) had been transferred to the bottom 10-feet (3 m) of the pile during the load test. We were unable to achieve the 150-ton (1,335 kN) target for the load test at this location.

Test Pile TP-2A was loaded to the load-test-frame capacity of 450 tons (4,000 kN). At 450 tons (4,000 kN) of applied load, the pair of strain gauges at 10-feet (3 m) from the pile tip recorded the same strain as recorded by the pair of strain gauges 2-feet (0.6 m) from the top of the pile (gauges closest to the applied load) indicating that 168 tons (1,495 kN) had been transferred to the bottom 10 feet (3 m) of the pile during the load test. The friction reducing jacket on the core beam in Test Pile TP-2A contributed to achieving a transfer of over 200% of the proposed design load (or 150-ton [1,335 kN] test load).

We compared the pile movement and strain recorded during both load tests. The results are similar in both tests, thus validating the results of the load test on Test Pile TP-1A. The load-movement plots for the test piles are presented in Figure 9.

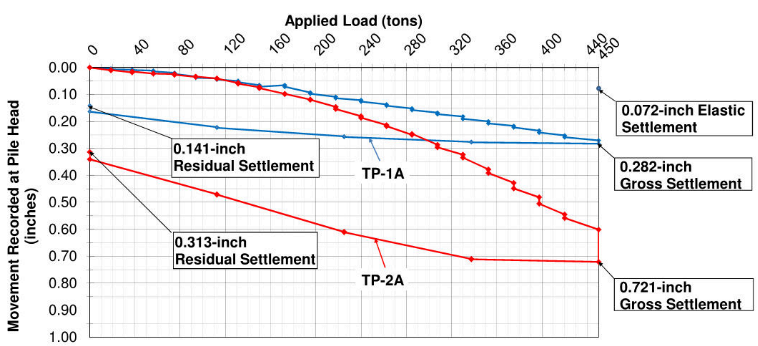

Figure 9. Test Load-Deflection Plot

We compared the axial stiffness of the pile based on the load transfer to the bottom 10-feet (3 m) of the pile and the calculated bond stress along the sides of the test piles based on the results of each load test. The calculated axial stiffness of the pile ratio (K=Load/Strain) values for each test pile are within about 10-percent of each other at about 77-tons (685 kN) transferred to the bottom 10-feet (3 m) of the pile ($K_{TP\text{-}1A,\ 77\text{-}ton}$=6.7 tons/inch and $K_{TP\text{-}2A,\ 77\text{-}ton}$=7.2 tons/inch) and at about 150-tons (1,335 kN) transferred to the bottom 10-ft of the pile in Test Pile TP-2A ($K_{TP\text{-}2A,\ 150\text{-}ton}$=6.9 tons/inch). The similar axial stiffness of the piles up to a load of 150-tons (1,335 kN) are indicative that Test Pile TP-1A performed similarly to Test Pile TP-2A. The difference in the axial stiffness of the bottom 10-feet of the test piles could be a result of varying conditions in the glacial till; for example, the concrete of the secant bonding to more boulders and cobbles in one pile compared to the other.

We used the results from the load tests to calculate the ultimate bond stress between the secant pile and the glacial till as about 35 psi (240 kPa) at Test Pile TP-1A and 30 psi (205 kPa) at Test Pile TP-2A. The higher calculated bond stresses at Test Pile TP-1A compared to TP-2A indicate that the bottom 10-feet (3 m) of Test Pile TP-1A can support 200% of the proposed design load.

The load tests confirmed that an allowable axial compressive load of 75 tons (665 kN) can be applied to a perimeter secant pile. The calculated spring constant and bond stress between the soil and the pile in Test Pile TP-1A in relation to Test Pile TP-2A verified that Test Pile TP-1A has an allowable axial compressive capacity of 75 tons (665 kN).

CONCLUSIONS

Support of excavation construction was completed within the overall project schedule, which likely wouldn't have been possible without the use of the CCFA techniques, testing of sacrificial secant piles, and a robust design that gave the contractor flexibility in their installation methods. No soldier piles needed to be re-drilled, and less than 1.5% of the anchors and secant piles needed to be re-drilled because of installation problems or field conditions.

Below-grade construction caused minimal disturbance to the neighboring properties. Vibrations in the neighboring buildings remained below ½ inches per second (12.7 mm/sec) during support of excavation installation. Lateral movement of the secant walls was less than ¼-inch (6.4 mm), which agreed with the FEM models. Movement of neighboring buildings measured approximately $^{1}/_{8}$-inch (3.2 mm) or less. Below-grade construction for the Center for Community Health project demonstrates that lot line construction for deep excavations can be achieved in sensitive urban environments with a support of excavation design that provides robust excavation support, substantial foundation support, and allows the contractor flexibility to use innovative installation methods.

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Evaluating and Managing Risk: Replacement of the Brooklyn Queens Expressway (BQE) Connector for the Kosciuszko Bridge in New York, New York

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ABSTRACT

The Skanska/Kiewit/ECCO III team (SKE) in association with HNTB and Haley & Aldrich (HNTB/H&A) was awarded the design-build contract for Phase I of the Kosciuszko Bridge Replacement Project. HNTB/H&A was responsible for design of the Brooklyn connector which includes the construction of new staged filled embankments utilizing prefabricated modular retaining walls (T-Walls). The indicative plans called for the new alignment to be constructed beneath the existing, in-service, column-supported Brooklyn Queens Expressway (BQE) using lightweight fill to avoid potential settlement impacts. The design-build team elected to use normal weight fill which led to unique design challenges of low overhead construction conditions and mitigation of potential construction impacts to the BQE on settlement sensitive shallow foundations. The subsurface investigation undertaken, derivation of soil parameters, and settlement assessment of the soils supporting the existing BQE and the deformation of the viaduct is discussed. A construction monitoring program was used to assess settlement predictions as they relate to the true behavior of the soils and structure.

INTRODUCTION

Skanska-Kiewit-ECCO III Joint Venture, in association with HNTB was contracted by the New York State Department of Transportation (NYSDOT) to design and construct the Phase 1 replacement of the Kosciuszko Bridge Design-Build (DB) Project. Phase 1 of the project involves the design and construction of the new eastbound structures of Interstate 278 over Newtown Creek from Brooklyn to Queens. The project runs between Morgan Avenue in Brooklyn and the Long Island Expressway Interchange in Queens (approx. 1.1 miles [1.8km]) and includes the demolition of the existing structures. The new cable-stayed bridge was constructed parallel to and on the eastbound side of the existing structures and carries both eastbound and westbound traffic until Phase 2 (not part of this contract) is completed, which will include a second cable-stayed bridge in the footprint of the existing Kosciuszko Bridge. The second bridge will carry westbound traffic and allow the bridge built in Phase 1 to carry eastbound traffic only.

This paper presents the challenges associated with the design of the Brooklyn Connector (Figure 1) detailing the design team's approach to confront the impacts and managing the risks associated with the placement of up to 45ft [13.7m] of fill which was to be asymmetrically placed in stages adjacent to and below the existing BQE which must remain in service during construction.

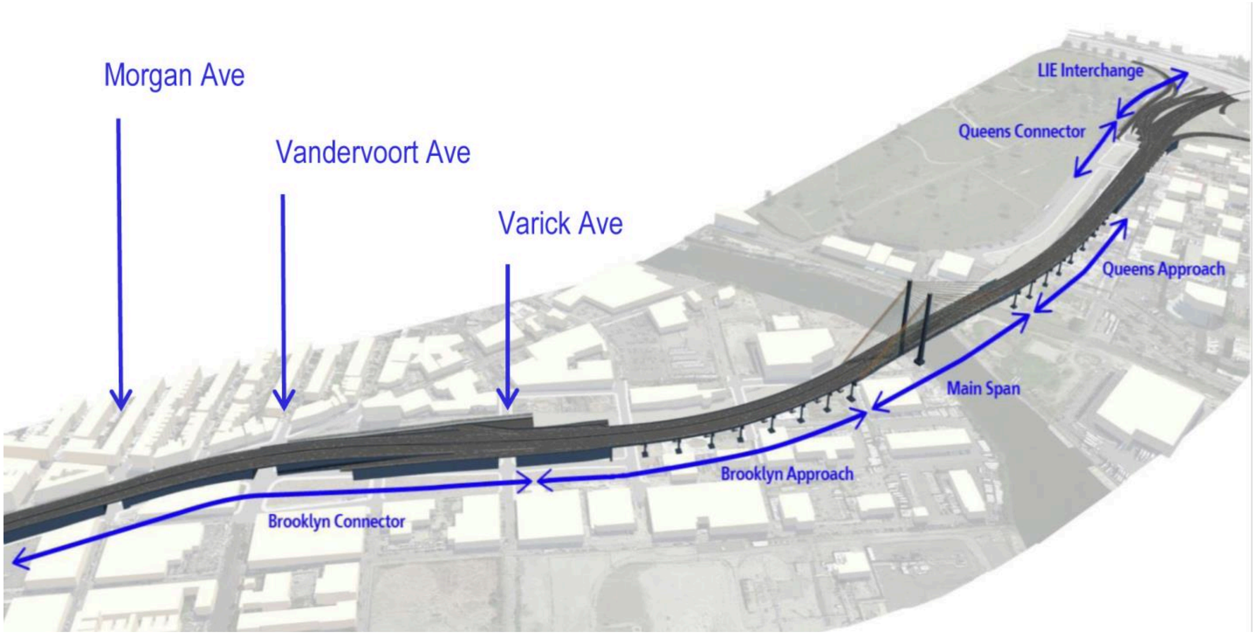

Figure 1. Project Site Overview

STRUCTURE SETTING

The existing Kosciusko Bridge viaduct extends from the Meeker Viaduct in Brooklyn to 54th Street in Queens, New York. The Brooklyn Connector that extends from the Meeker Viaduct to the Brooklyn Approach was completed in 1939 and modified in 1971. The Brooklyn Connector consists of 78 rigid concrete spans supported by concrete piers on spread footings that are founded approximately 10 to 15ft [3.0 to 4.6m] below the existing grade on the natural silty sand deposits. Portions of the connector also contain concrete closure walls with a brick veneer.

The connector structures are typically composed of a series of 3-span continuous rigid concrete frames in the longitudinal direction and two 2-span bents in the transverse direction. .

There is a longitudinal expansion joint between the eastbound and westbound halves, down the centerline of the bridge, with the center pair of columns beneath the joint supported on a common footing. The transverse interior bents consist of a single row of columns and cap beams for the interior bents and double columns and cap beams at the expansion (end) bents. The pairs of columns at the transverse expansion bents are supported on common footings. The original superstructure consisted of a 7in [17.8cm] thick deck supported on longitudinal stringers (ribs) that framed into the transverse cap beams and was all cast-in-place concrete construction.

At the start of construction, the existing structure was found to be in generally good condition. The deck exhibited some map cracking and efflorescence; this was more prevalent on the eastbound side than westbound. There also was some minor deterioration and spalling of the columns, mostly at the lower portion of the columns. The cap beams generally appeared to be in sound condition.

The site is underlain by Cambrian-Ordovician Age metamorphic bedrock of the Hartland Formation, primarily gneiss of varying quality with a zone of highly weathered and decomposed rock of varying thickness overlying the competent rock. Generally, the bedrock surface dips to the southeast. Overlying the bedrock is Cretaceous Raritan Formation soils, which exist as a confining layer, and are generally described as a light to dark gray, brown-red, pink red and gray white clay, silty clay and clayey to silty fine sand.