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.

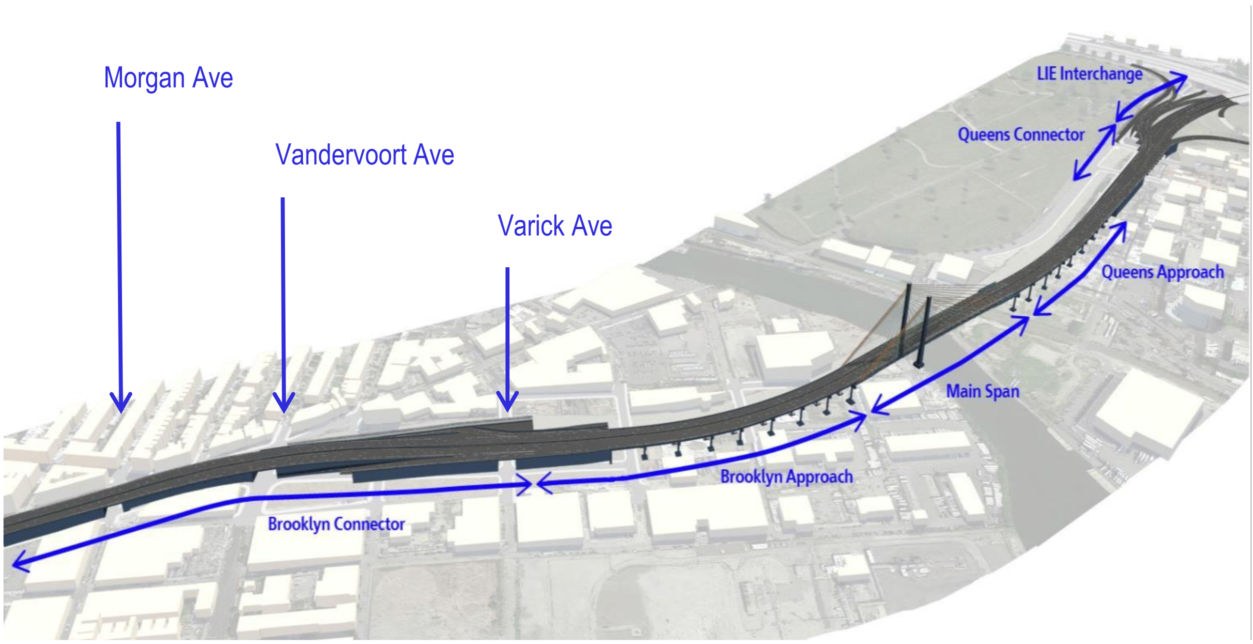

Figure 1. Project Site Overview

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.

The advance of the Wisconsin glaciation roughly 15,000 years ago deposited Harbor Hill moraine across the site. The till of this moraine is comprised of an unsorted mixture of clay, silt, sand, gravel, cobbles and boulders. The till is overlain by organic soils surrounding the Newtown Creek which extend both eastward and westward along the Brooklyn and Queens Approaches. Surficial soils are variable in the upper 10 to 30ft [3.0 to 9.1m] of the profile due to miscellaneous urban fill material that is comprised of a mixture of silt, sand, gravel and construction debris (e.g. brick, wood, trash, concrete and other materials).

The groundwater table was found within the upper glacial deposits upland and just below the fill as it approaches the creek. The groundwater ranges in depth from approximately 10 ft [3.0m] below the existing ground surface near Newtown Creek, to 20 to 40ft [6.1 to 12.2m] below the existing ground surface as the existing grade rises to the east and west of the creek.

A generalized longitudinal subsurface profile interpreted parallel to the bridge alignment was developed (excerpt shown below in Figure 2). Subsurface conditions generally consist of strata including, in descending order: Fill, Glacial Sand, Glacial Silt & Clay, Raritan Clay, Decomposed Rock, and Bedrock.

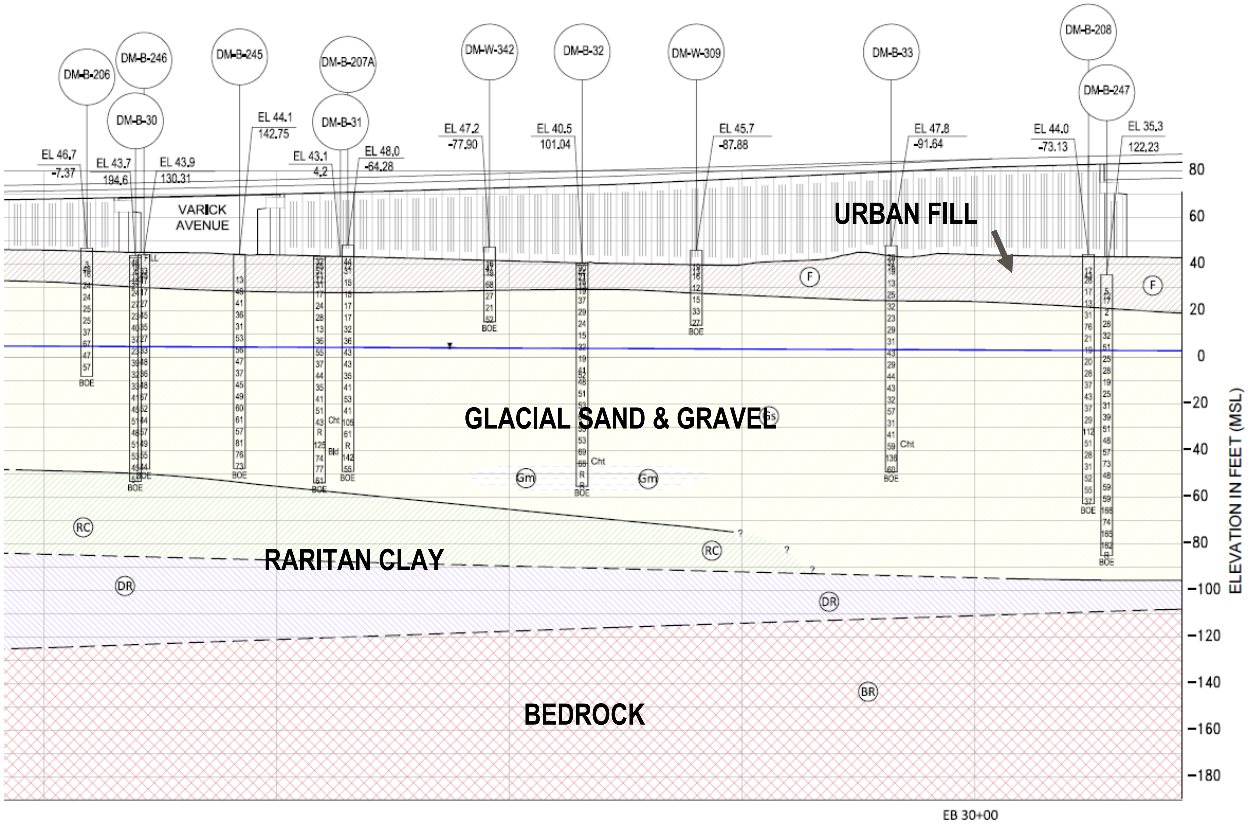

Figure 2. Typical Subsurface Profile

Based on the stresses induced from the proposed wall and embankment loads, induced settlements below approximately 70ft [21.3m] are insignificant. Therefore, strata below the Glacial Sand were not impacted by the proposed wall and embankment construction and have not been described in this paper.

Settlement evaluation - Geotechnical

Overview. The proposed alignment which is retained by T-Walls to the south and geosynthetic reinforced earth system (GRES) walls to the north, was ultimately constructed beneath or adjacent to the existing BQE or while that roadway carried traffic. The new embankment ranged in height from approximately 13 to 45ft [4.0 to 13.7m]. Construction beneath the existing BQE, which is supported on shallow foundations, presented some unique design challenges associated with work in low overhead conditions and potential impacts to existing structure. Because the DB team elected to use normal earth fill rather than lightweight fill (EPS) it was up to designer to evaluate impacts and prove that this approach could work safely. Geotechnical assessment of the soils supporting the existing BQE were a critical aspect of this design, as their behavior when subjected to additional stress beyond their current state drives the deformation of the viaduct. In addition, the Designer performed analyses to assess the existing structure and its tolerance to both total and differential settlement.

The geotechnical engineers approached this design initially through computations based on elastic settlements with soil parameters derived from SPT investigations. This approach was deemed inadequate to define and mitigate the obvious risks due to the high amount of gravel encountered and inaccuracy of SPT correlations to elastic soil parameters. These challenges led to the subsequent advance of a CPT/SCPT sounding program to better define the elastic parameters of the overlying urban fill and the glacial deposits below which encompass the zone of interest. To further validate and calibrate design methodologies the Designer also completed a static settlement load test at the southeast corner of Varick Avenue and Cherry Street. This in-situ load test and the adjacent SCPT soundings provided an opportunity for the design team to calibrate estimated elastic parameters.

Settlement evaluation addressed both the performance of the proposed T-Walls as well as the impacts to the adjacent facilities as previously noted. The progression of the Designer's assessment followed the design steps noted below:

- Evaluate soil settlement parameters from subsurface information gathered by both previous and current investigations.
- Establish staging timeframes and sequencing.
- Conduct settlement analyses for embankment configurations and construction staging.
- Estimate lateral extent of settlement induced by proposed construction.
- Develop a construction monitoring plan and a strategy to incorporate "hold points" in our approach allowing the Designer to compare deformations experienced to settlements predicted and revise the build for subsequent stages as deemed appropriate.
- Determine where the contingency plan needs to preemptively be implemented where differential settlements have the potential to exceed tolerances.

Settlement Parameter Determination. Settlement analysis performed for the T-Walls and embankments where fill heights reach 45ft [13.7m] was a critical component and is major focus of this paper. Of primary concern was the potential for impact to the existing BQE foundations which remained in service as embankment fills and T-Walls were constructed. As the alignment progresses east of Vandervoort Ave. the proposed embankment diverges to the south of the existing viaduct. The diverging alignment means diverging fill placement creating differential settlement concerns associated with asymmetric loading of adjacent foundations.

Since the predominant strata being stressed is granular in nature, settlement was anticipated to be elastic and governed primarily by the elastic modulus (Es) of the soil. Several methods to estimate the soil modulus from SPT and CPT tests were employed. Ultimately the design relied upon methods of correlating Es values from CPT soundings. Induced settlements from the proposed wall and embankment loads are insignificant below approximately 70ft, based on the stresses induced. Therefore, consolidation settlement in the underlying Raritan clay strata is not subjected to changes in stress that would induce settlement given the proposed wall and embankment loads.

Given the sensitivity of the existing structure to differential settlement and the significant consequences that unanticipated differential settlements may cause, a rigorous assessment was made of potential global variation as well as local variation of soil moduli. The Es value was the single most important geotechnical parameter in this assessment and the design team took great measures to develop this parameter.

Several published relationships between elastic modulus and a range of assumptions and conditions were reviewed and assessed. Baldi et al., (1989) correlates secant elastic modulus for an average axial strain of 0.1% for a range of stress histories and aging based on the normalized cone resistance. Additionally, Robertson (1991) established methods to estimate the equivalent elastic modulus based on CPT results in sands based on the degree of loading and cone resistance, with corrections for aging. Although both these methods are valid, they were dismissed in favor of more recent methods proposed by Robertson (2009), in which estimates of elastic modulus are based on correlations to shear wave velocity and the subsequent relationships between shear wave velocity, shear modulus and elastic modulus.

The accumulated experience over the last 25 years with SCPT results has led to the establishment of relationships between cone resistance and shear wave velocity, Vs, for a wide range of soils using the CPT SBTn chart as a basis. Robertson et al. (1992) developed a set of contours on the normalized SBTn Qtn-Fr chart of normalized shear wave velocity based on over 100 SCPT profiles combined with available published data. In order to verify the use of Robertson's method, results from six CPT soundings located along the Connector were correlated to shear wave velocity. Shear wave velocity correlations were averaged from the ground surface to 20ft [6.1m] and from 20 to 35ft [6.1 to 10.7m] to encapsulate the fill and glacial sand respectively.

The results of the CPT correlated shear wave velocities were averaged and compared to measured average values of shear wave velocity determined from PS logging in the fill and glacial sands performed at other locations on the project site as part of the pre-award subsurface investigation. The shear wave velocity values developed from CPT correlations are in general agreement with measured shear wave velocity values in both the fill and cohesionless glacial sands based on this preliminary assessment method.

An additional assessment of measured shear wave velocity relative to correlated shear wave velocity was performed near Varick Avenue where a field load settlement test was performed. A seismic cone was used to obtain measurement of shear wave velocity at 5ft [1.5m] intervals in this area. The correlated values were plotted verses depth using the previously-discussed methodology and are shown in the Figure 3 below. Both correlated shear wave velocity data and measured shear wave velocity results were assessed over ten-foot intervals.

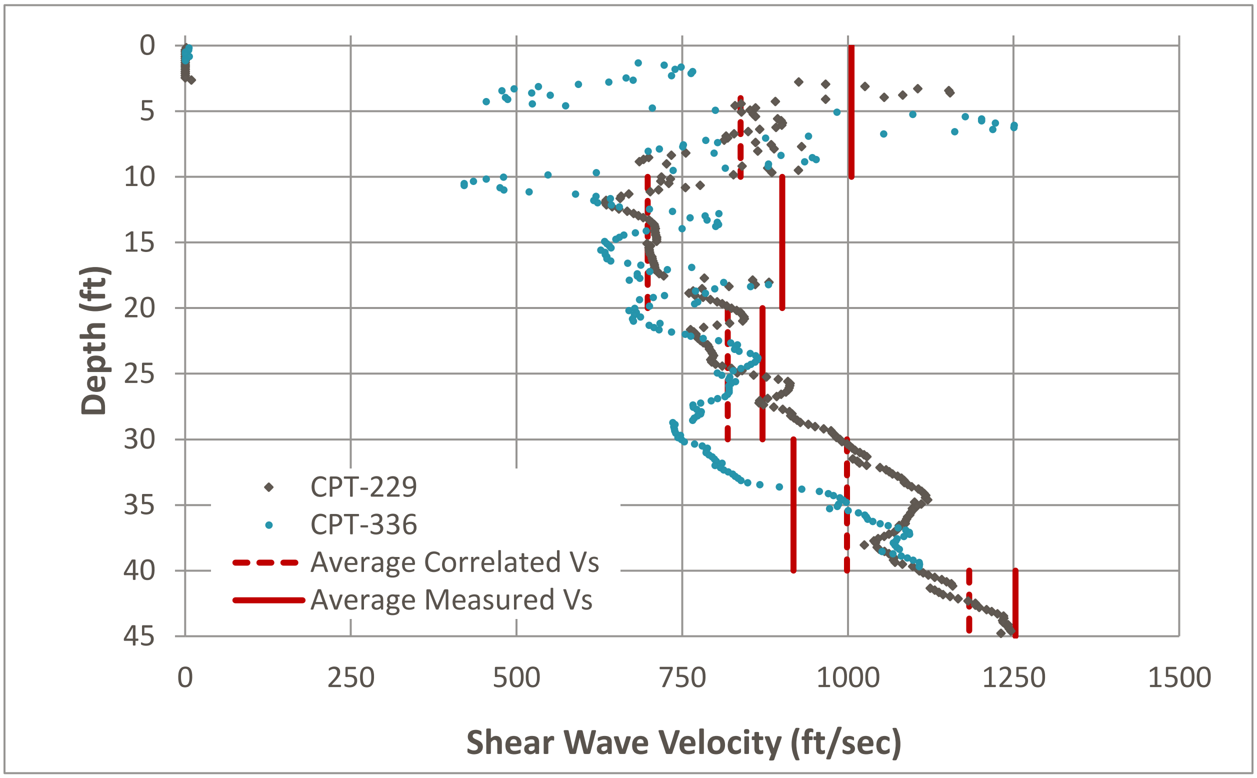

Figure 3. Correlated Shear Wave Velocity relative to Measured Shear Wave Velocity from SCPT at Field Settlement Load Test

The results indicate that the SCPT to shear wave velocity correlations generally underestimate the shear wave velocity by as much as 22% in the fill and upper regions of the glacial sands relative to the measured values. The correlation appears to overestimate values for shear wave velocity relative to the measured values by approximately 9% between 30 and 40ft [9.1 to 12.2m]. Below 40ft [12.2m] the correlation appears to underestimate the measured shear wave velocity by approximately 6%. Overall, using the shear wave velocity correlations specified above appear to be somewhat conservative in relation to estimating elastic modulus for settlement analysis.

A settlement load test was performed at the southeast corner of Varick Avenue using concrete blocks stacked approximately 15ft [4.6m] high and loaded in stages. Loading of the soil occurred in three stages consisting of a 450psf [21.6kPa] load, a 1350psf [64.6kPa] load and finally a 2250psf [107.7kPa] load. Settlement was measured over time as the load was increased. In order to verify the recommended correlation methods, settlement was monitored using three survey points mounted to settlement plates bearing on the existing ground surface under the load. The photo in Figure 4 depicts the final load stage.

Figure 4. Settlement load test at the southeast corner of Varick Avenue

To verify the correlation methods and calibrate the appropriate G/Go ratio, settlement was estimated using the program Settle3D by RocScience. Parameters corresponding to G/Go ratios of 0.2, 0.25 and 0.3 were assessed. Given that the purpose of the exercise was to match the results of the field settlement load test using correlated data from CPT soundings at the location of the test, average values of shear wave velocity were used in the assessment. The available CPT data was assessed, and parameters were developed for three different sections of viaduct based on common soil properties.

Settlement Analysis. In assessing impacts to the existing viaduct as well as the proposed T-Walls, lower bound, average and upper bound soil parameters were used to determine the anticipated total settlement and differential settlement (at grade for T-Wall assessment and at depth for existing foundation assessment) for these three regions. To capture the condition where soil parameters vary underneath adjacent existing columns, differential settlements were assessed using the unlikely scenario where lower bound parameters were present underneath one column and upper bound values were present underneath an adjacent column. Upper bound and lower bound settlement estimates were conducted at each discrete existing column location to facilitate this assessment.

The software program Settle3D version 2.011 was used to evaluate the settlement of the T-Walls and embankments. A staged analysis was performed to model the fill placement. A benefit of performing the staged settlement analysis is that it facilitates the evaluation of the cumulative total and differential settlement at any particular step as well as the incremental settlement between steps. The results of the settlement analyses for the embankment between Vandervoort and Varick Avenues are presented in

Figure 5 and the table below. The table presents settlement at the existing footing depths, which corresponds to the approximate bearing level of the existing spread footings that support the existing bridge. The majority of the existing footings are approximately 12ft [3.7m] below existing ground surface, however according the structure as-builts some footings range from about 9ft [2.7m] below grade to 24ft [7.3m] below grade. The settlement model was run to assess the impacts of these differing depths and the data provided reflect these results.

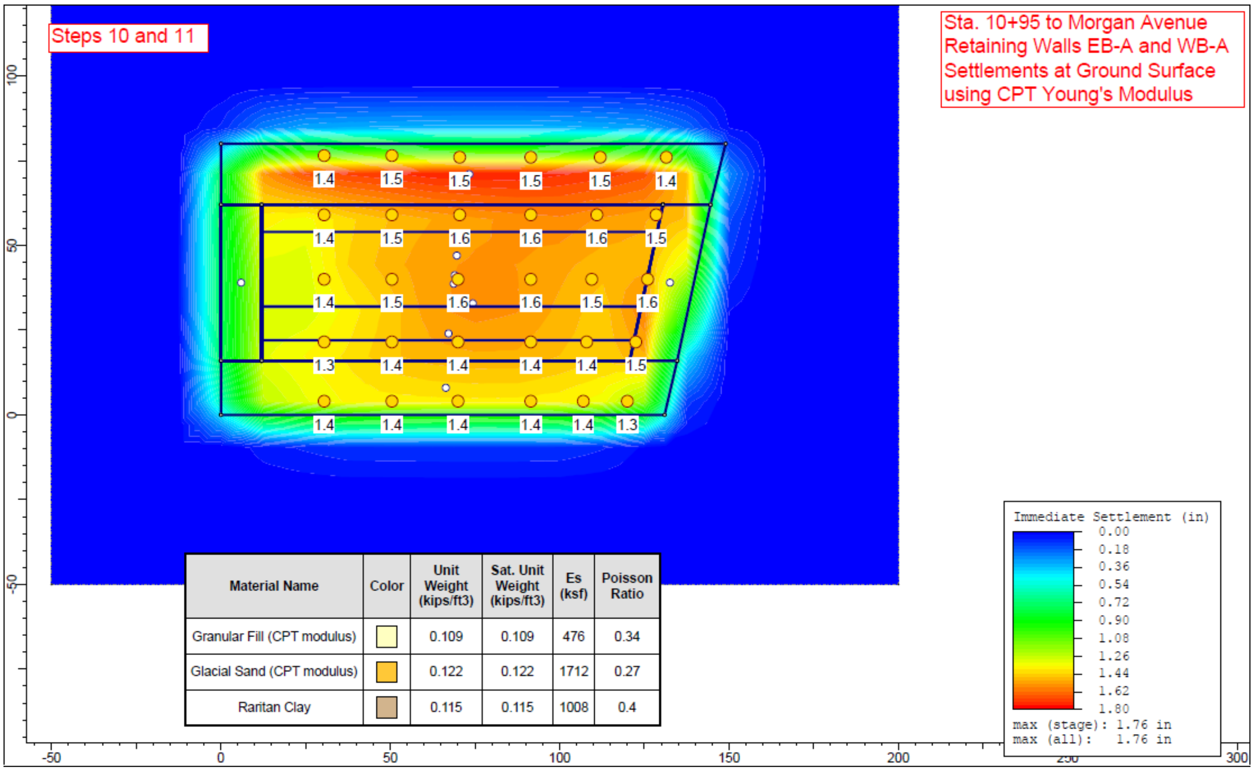

Figure 5. Typical Output of Settle3D modelling

For the Vandervoort to Varick section the table below presents maximum differential settlements between any two columns in the associated design area assuming that variable soil conditions exist at adjacent columns.

VARICK TO VANDERVOORT						
Stage	**Maximum Differential Settlement Between Columns (in) [cm]**				**Max. Total Settlement (in) [cm]**	
	Lower Bound vs. Lower Bound	**Lower Bound vs. Upper Bound**	**Upper Bound vs. Upper Bound**	**ALL CASES**	**Lower Bound Modulus**	**Upper Bound Modulus**
1D	0.46 [1.2]	0.79 [2.0]	0.12 [0.3]	0.8 [2.0]	0.88 [2.2]	0.21 [0.5]
2A	0.46 [1.2]	0.62 [1.6]	0.30 [0.8]	0.6 [1.5]	0.90 [2.3]	0.65 [1.7]

VARICK TO VANDERVOORT						
Stage	Maximum Differential Settlement Between Columns (in) [cm]				Max. Total Settlement (in) [cm]	
	Lower Bound vs. Lower Bound	Lower Bound vs. Upper Bound	Upper Bound vs. Upper Bound	ALL CASES	Lower Bound Modulus	Upper Bound Modulus
2D	0.70 [1.8]	0.76 [1.9]	0.51 [1.3]	0.8 [2.0]	1.23 [3.1]	0.90 [2.3]
3A	0.89 [2.3]	0.89 [2.3]	0.56 [1.4]	0.9 [2.3]	1.38 [3.5]	0.90 [2.3]

Table 1. Maximum differential settlements between any two columns and total settlement

The results of the analysis performed suggests that the estimated maximum differential settlement for the existing footings, assuming lower bound and upper bound soil parameters are present at adjacent existing columns, is 0.9in [2.3cm]. Maximum total settlement of the existing bridge footings was expected to be about 1.38in [3.5cm].

STRUCTURAL IMPACTS

The existing structure was analyzed to evaluate its capacity to tolerate differential settlements during staged construction of the new retained fill structures. This analysis was performed for typical intermediate and expansion bents which are typical for the existing structure from Vandervoort Avenue to Varick Avenue and from Varick Avenue to Stewart Avenue in terms structural configuration. The existing bent heights vary throughout the region from Vandervoort Avenue to Varick Avenue and from Varick Avenue to Stewart Avenue. The bent heights analyzed are the shortest intermediate and expansion bents in this region. This is generally conservative for most of the bents in this region because shorter bents are inherently stiffer than taller bents. Hence, shorter bents develop higher shear forces and moments when subjected to relative displacements than taller bents.

Nonlinear static analysis was performed for each bent in which individual columns were displaced (settled) incrementally. The nonlinear static analyses determined that the existing structure has sufficient displacement capacity to accommodate relative foundation settlements in excess of the anticipated geotechnical settlements without structural modifications. However, the existing bent cap beams were determined to have inadequate shear capacity for the combination of settlement with dead and live loads. Therefore, a retrofit scheme was designed and detailed to provide additional shear capacity for the existing bent cap beams. The retrofit consists of Fiber-Reinforced Polymer (FRP) wrapping which will be installed prior to the start of fill placement.

In summary, with the FRP wrapping retrofit of the existing bent cap beams, the existing structure is shown to have sufficient capacity to accommodate dead loads and HL-93 live load (with impact) in conjunction with relative foundation settlements that are equal to or greater than the anticipated geotechnical settlements.

SETTLEMENT MONITORING

Instrumentation and Monitoring Plan. Recognizing the importance to life-safety and essential service the BQE provides to daily New York City traffic patterns, the risk tolerance for a potential shut-down of the corridor was low. All parties involved recognized that a monitoring program that relies solely on the exceedance of an absolute value would not be satisfactory. Rather, an observational approach that considers both monitoring thresholds and an assessment of the potential for future deformation trends was used to assess the response of the existing structure to surcharge loads. The approach presented herein was designed to keep the DB team informed as to the results of future actions and allows for adjustments to the program on a pro-active, rather than re-active basis.

Instrumentation was installed within the project limits and the surrounding neighborhoods and data was collected to evaluate the influence of construction. The parameters monitored in the program were absolute and relative settlement, vibrations, and a limited amount of tilt and subsurface settlement. Monitoring equipment included redundant optical monitoring systems (a series of automated total stations (AMTS) monitoring reflective mini-prisms and a secondary optical camera system capable of measuring deformations by pixel differentiation), in-ground settlement sensors, vibrating wire tiltmeters, and seismographs.

Data was uploaded to a server that was available for NYSDOT, SKE, and Designer access. Observational approach methods were employed as the construction proceeded and monitoring data was observed, such that frequency of measurement and types and numbers of instruments were adjusted, either up or down as appropriate, to meet the needs of the activities being performed. The observational approach also allowed for adjustments to the construction operations. Receptor criteria were set based on the generally anticipated conditions expected.

As a means of further validating the analyses, the Designer assessed the observed settlement the existing viaduct and support piers experienced and the movement of the ground surface. "Construction Hold Point" at each incremental 5ft fill height level allowed the Designer an opportunity to review the data collected and assess the predicted versus actual deformation and, if required, adjust the build approach accordingly to maintain settlements within the tolerances described above.

Monitoring thresholds were established based on the structural tolerance of the existing structures. Data was reviewed, analyzed, and interpreted on an on-going basis with a focus on defining characteristic trending of structural response to filling surcharge loads. Trend lines were used to predict the future effect of additional fill placement and to prevent placing fills in excess of that which would exceed thresholds. The purpose of tracking the trends was to inform the Designer and the builder of possible threshold exceedances before they actually occur.

Monitoring Approach and Analysis Results. Given the high importance of and concern for the existing structure, the option to selectively monitor specific locations was not acceptable and so nearly every column of the Connector was monitored. Due the geometric arrangement of the Connecter which included 78 bents with between 6 to 8 columns each, there were on the order of 500 mini-prism measurements being made and nearly double that number of differential settlement measurements points that required continual review by the Designer. Figure 6 shows a typical example of AMTS differential measurement points as reported via a web-based server (Argus) for both the transverse and longitudinal column pairs. Within this 8-bent length of the Connector there were 129 differential measurements made which represented approximately 92% coverage of possible combinations.

Figure 6. Typical Differential Settlement Monitoring Plan of Brooklyn Connector between Varick and Stewart Avenues

The number of data made it necessary to develop a practical approach to assess the measurements. The approach taken was a collaborative effort developed between the Designer, SKE, and SKE's instrumentation subcontractor, and is described herein with an example of data obtained for the portion of the Connector located between Varick and Stewart Avenues in Brooklyn.

The first challenge was maintaining accurate records of embankment fill heights which required daily manual survey and reporting by SKE to the Designer. On a daily basis, SKE issued a snapshot report of the Argus data which provided a tabulation of daily absolute measurements and averaged differential values between columns. The tabulation was hot-color coded as values approached threshold levels. These tabulations allowed the Designer to have an overview of areas where differential movements were relatively greater than other areas and allowed the Designer to focus review efforts into areas with greater potential concern.

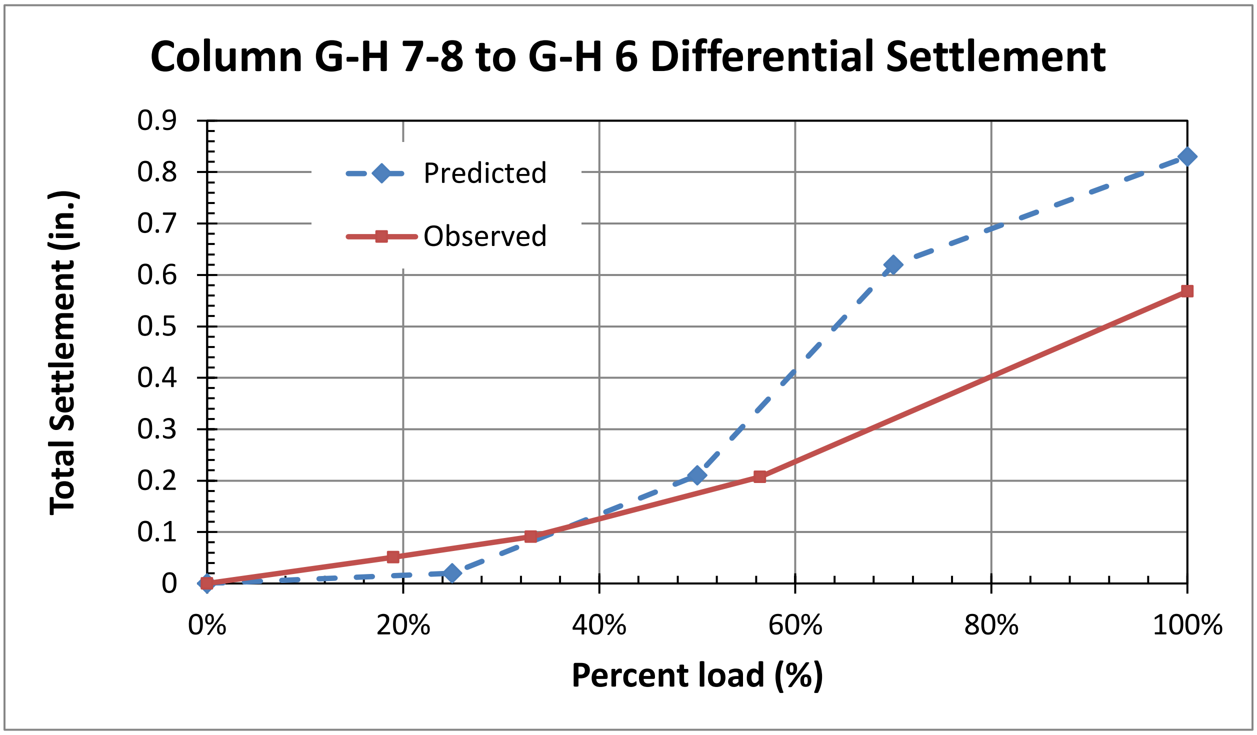

Figure 7. Typical Observational Trend Analysis of Differential Settlements at a Select Column of Brooklyn Connector between Varick and Stewart Avenues

Based on the daily report and tabulated surcharge data, the Designer was able to generate specific trend plots, as originally proposed in the monitoring plan, to determine the relative performance of a given column pair compared to the predicted movements. Figure 7 shows a typical plot for a column pair in the Connector between Varick to Stewart Avenues. This observed trend line is a typical example of a pair that initially began to settle more than predicted, but as greater loads were placed, the settlement trend began to run under the predicted level. Although this column pair eventually exceeded the Alert threshold level, it was apparent by trend analysis that it would not reach the Action threshold. By this method, the Designer was able to agree to allow filling to continue without the need for SKE to begin preparations to implement remedial actions.

Conclusions

The monitoring program was used to validate the predictive models, identify areas of concern, provide a rational means for decisions on construction process, maintain contractor's construction schedule and, most importantly, protect the public and health of the city's regional traffic flow. The monitoring program was developed with comprehensive and redundant measurement systems driven by the structural characteristics and importance of the Connector. In order to keep the construction operations moving fluidly and reduce the "surprise" element of reaching critical thresholds requiring unexpected work stoppages, an observational approach including trend analyses was adopted. The trend analyses were used to determine if the settlements compared to the relative surcharge were below, as-expected, or above predicted levels. More importantly, the trend analysis was used to determine if the rate of settlements were accelerating or decelerating giving the Designer the ability to predict if critical thresholds would ever be reached.

Overall, the monitoring program in conjunction with the predictive analyses, gave confidence to the Designer that the filling operations could be controlled to maintain the structural integrity of the Approach, gave the SKE advance notice of where work could proceed and where remedial actions might be needed so that changes to the schedule did not come as a surprise, and gave NYSDOT the assurance that traffic patterns and life safety could be maintained on the existing bridge.

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