ABSTRACT
The Metropolitan Transportation Authority (MTA) of New York is presently upgrading their track alignments for passenger and freight rail services as part of their Long Island Railroad East Side Access Transportation Corridor project. In the Harold Interlocking portion of the Sunnyside Rail Yard located in Long Island City, New York, new utilities and below-grade structures are being installed within the alignment and adjacent to existing MTA and Amtrak rail lines. These include new sewer, drainage, electrical, telecommunication, power substations, track approach structures, passageway structures, bridge abutment extensions, retaining walls and other below-grade construction installed in excavations up to 30 ft. to 40 ft. deep using support of excavation designs including steel sheet piling, soldier pile and lagging with levels of internal and external bracing.

Engineering support of excavation designs were performed using finite element models to prepare efficient designs based on displacement-based contract document criteria. Design/construction considerations included designing support of excavation for loose and soft soils, railroad dynamic loading, installation of support of excavation into dense glacial soils, high groundwater, installation of support of excavation under existing railroad alignments, and designing for small rail alignment displacement tolerances. An extensive instrumentation program was implemented to monitor the performance of the rail alignments adjacent to the support of excavation-supported excavations for confirmation of the finite element modeling and for compliance with the rail displacement limits required by the project.

INTRODUCTION
The East Side Access (ESA) project in New York City will connect the Long Island Rail Road’s (LIRR) Main and Port Washington lines in Queens to a new LIRR terminal beneath Grand Central Terminal in Manhattan (Figure 1). The new connection will increase the LIRR’s capacity into Manhattan, dramatically shorten travel time for Long Island and eastern Queens commuters traveling to the East Side of Manhattan, and
provide a new commuter rail station in Sunnyside, Long Island City, located in the borough of Queens.

The Harold Interlocking, located in the Sunnyside Rail Yard in Long Island City (Figure 2), is a large rail yard to manage rail traffic including freight and passenger of the LIRR, Amtrak and others. The Metropolitan Transportation (MTA) of New York is rehabilitating and upgrading these track alignments in the Harold Interlocking as part of their Long Island Railroad East Side Access Transportation Corridor project. Associated infrastructure improvements to the Harold Interlocking and Sunnyside Rail Yard will include new utilities and below-grade structures including new sewer, drainage, electrical, telecommunication, power substations, track approach structures, passageway structures, bridge abutment extensions, retaining walls and other below-grade construction located adjacent to existing railroad tracks, buildings, bridge foundations, roadways, utilities and other existing structures. The excavations for construction of these improvements were made up to 30 ft. to 40 ft. deep within close proximity to existing track alignments, below groundwater, through thick fills and loose and soft soils, requiring temporary support of major utilities crossing the excavation, and design of temporary bridges to support drilling equipment. Engineering designs for support of excavation were performed using finite element solutions to prepare efficient designs based on displacement-based contract document criteria.

SITE CONDITIONS AND DEVELOPMENT HISTORY
The Harold Interlocking and Sunnyside Rail Yard project site is a vast rail yard used for freight and passenger rail traffic from Long Island and Connecticut into Manhattan. The site is located in a deep depression (up to 60 ft deep) in the heavily urban area landscape to allow transition from the elevated track alignments from the east to the tunnel entrances at the west end of the site for passage under the East River into Manhattan. Historical pre-development site conditions consisted of relatively level topography with rolling hills and natural water courses promoting surface drainage Figure 3). One such natural water course originally passed through the project site.

GEOLOGY
The site subsurface conditions are controlled by the historical railroad development and the site geology. The historical railroad development created excavations below the general site grades to meet tunnel inverts and some filling in other areas to above general site grades. The fills are variable in thickness and composition, sometimes containing building debris and other man-made materials. The fills are underlain by thick, compressible organic deposits, varying from organic silts to peat or glacial soil deposits which form the bottommost layer above bedrock. The glacial soil deposits vary from dense, well-graded glacial till to coarse-grained outwash deposits to fine-grained silty and clayey glaciolacustrine deposits. Depth to top of rock varies but is generally 20 ft. to 80 ft. below existing ground level at the site.
DESIGN SOIL AND ROCK PARAMETERS

Based on the results of test borings and laboratory testing performed on recovered soil and rock samples, the following design soil and rock parameters were developed and used in modeling the performance of the support of excavation systems:

Table 1 - Engineering Design Parameters for Soil and Rock

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Eff. friction angle $\varphi'$ (deg.)</th>
<th>Undrained shear strength $S_u$ (psf)</th>
<th>$q_u$ (psi)</th>
<th>Coeff. of earth pressure at-rest $K_o$</th>
<th>Unit weight $\gamma$ (pcf)</th>
<th>Eff. $\gamma'$</th>
<th>Young's Modulus E (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Miscellaneous Fill</td>
<td>30-33</td>
<td>-</td>
<td>-</td>
<td>0.5-0.7</td>
<td>125</td>
<td>63</td>
<td>2.0-2.5</td>
</tr>
<tr>
<td>Engineered Fill</td>
<td>34</td>
<td>-</td>
<td>-</td>
<td>0.5</td>
<td>125</td>
<td>63</td>
<td>6.9</td>
</tr>
<tr>
<td>Organics</td>
<td>-</td>
<td>50-500</td>
<td>-</td>
<td>0.65</td>
<td>85-110</td>
<td>23-48</td>
<td>-</td>
</tr>
<tr>
<td>Glacial Soil Deposits</td>
<td>38</td>
<td>-</td>
<td>-</td>
<td></td>
<td>135</td>
<td>73</td>
<td>12.5</td>
</tr>
<tr>
<td>Clay</td>
<td>32</td>
<td>2,500</td>
<td>-</td>
<td>-</td>
<td>130</td>
<td>68</td>
<td>6.9</td>
</tr>
</tbody>
</table>

SPECIFIED PERFORMANCE CRITERIA

**Allowable Deformations of Railroad Tracks.** Allowable deformations for the railroad tracks were defined by requirements given in the contract specifications. The allowable deformations depended on a number of geometrical considerations including the following items.

- runoff in 31 ft. of rail;
- deviation from uniform profile on either end of a 62 ft. chord;
- deviation from zero cross level on a tangent;
- reverse elevation on curves;
- difference in cross level between any two points less than 62 ft. apart;
- deviation from mid-ordinate from a 62 ft. chord;
- deviation from mid-ordinate from a 31 ft. chord on a curved track.
The allowable deformations for purposes of design are summarized as follows:

Table 2 - Allowable Deformations for Railroad Tracks

<table>
<thead>
<tr>
<th>Track Condition</th>
<th>Allowable Deformation (in.)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Curved Track or Switches</td>
<td>≤ 5/8</td>
</tr>
<tr>
<td>Straight Track</td>
<td>≤ 7/8</td>
</tr>
</tbody>
</table>

Note: *vertical and horizontal deformation

Minimum Factors of Safety on Stability. For temporary support of excavation supporting structures, railroads and utilities, the minimum required factors of safety on global stability were as follows:

Table 3 - Minimum Geostructural Factors of Safety

<table>
<thead>
<tr>
<th>Wall Type</th>
<th>Stability Type</th>
<th>Minimum Factor of Safety</th>
<th>Static</th>
<th>Seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>ODE</td>
<td>MDE</td>
</tr>
<tr>
<td>Type 1</td>
<td>Global and Internal</td>
<td>1.5</td>
<td>1.1</td>
<td>1.0</td>
</tr>
<tr>
<td>Type 2</td>
<td>Global and Internal</td>
<td>1.3</td>
<td>1.1</td>
<td>1.0</td>
</tr>
<tr>
<td>Permanent Wall</td>
<td>Global and Internal</td>
<td>1.5</td>
<td>1.1</td>
<td>1.0</td>
</tr>
<tr>
<td>Modular Wall</td>
<td>Sliding</td>
<td>1.5</td>
<td>1.1</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Overturning</td>
<td>2.0</td>
<td>1.3</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td>Bearing Capacity</td>
<td>3.0</td>
<td>1.1</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Slope Stability</td>
<td>1.5</td>
<td>1.1</td>
<td>1.0</td>
</tr>
</tbody>
</table>

DESIGN OF SUPPORT OF EXCAVATION
The deformation analysis of the support of excavation (SOE) was performed using the 2-Dimensional plane-strain finite element code Plaxis, Versions 8 and 9 (Brinkgreve et al., 2009; Figure 4). Plaxis is a finite element method (FEM) package that is specifically intended for deformation and stability analysis in geotechnical engineering. Plaxis is equipped with features that enable it to perform stability analyses taking into consideration all aspects of the cross section geometry including structural elements and user defined loading. The FEM analysis also takes into consideration stress redistributions from soil-structure interaction that are important for determining realistic earth pressure forces against retaining structures. The soils at the project site were modeled using the Plaxis Mohr-Coulomb model. This is a bi-linear elastic perfect-plastic soil model which is a combination of Hooke’s law and the generalized form of the Coulomb failure criterion (Brinkgreve, 2005). The model requires the following input parameters:
Elastic parameters from Hooke’s law: Young’s Modulus, E and Poisson’s ratio, ν;
Parameters from Coulomb’s failure criterion: friction angle, φ and cohesion, c.

In the Plaxis Mohr-Coulomb model, the stiffness behavior is modelled as linear elastic below the failure envelope. The model does not account for softening behavior after the peak strength has been reached. The Mohr-Coulomb model parameters used in the design are summarized in Table 1.
The fill and glacial deposits underlying the site were modeled as drained materials where no excess pore pressure generation takes place during loading. The fine-grained soils and rock were modeled as undrained materials where full generation of excess pore pressures take place during loading.
In the FEM model, retaining walls were modeled as elastic-plastic bending elements. Anchors and struts were modeled using elastic-plastic axial tension-only elements. Soil-structure slippage was modeled using interface elements.

Deformation Analyses. The deformation analysis starts with determining the initial stress conditions in the ground. The subsurface layers modeled in the finite element code were in accordance with information interpreted from borings and geotechnical laboratory tests. The initial horizontal stresses are computed from the self-weight of the soils and the coefficient of earth pressure at-rest, K₀ values shown in Table 1. All stresses within the soil mass must remain on or within the failure envelope defined by the Mohr-Coulomb strength parameters. The analysis steps through the construction staging and computes a new equilibrium stress condition for each stage. The results from the previous stage are used as the starting condition for the next construction step. The process of stepping through the construction staging in the analysis is performed until the final conditions are achieved (Figure 5).

Stability Analyses. Stability analysis is performed to confirm that the design meets the minimum factors of safety presented in Table 3. The design method selected uses FEM computations to model the service condition and the ultimate load conditions at critical sections. The geometry and analysis results at the critical sections are shown on the design drawings. Critical sections are selected using the following conditions:
- Maximum loading conditions
- Most restrictive deformation criteria
- Most critical geometry
- Typical and representative sections

The following items are considered in the stability analysis performed using FEM.
- Redistribution of stresses against the structure
- Yielding and redistribution of stresses to form a failure surface
- Limited resistance available from structures (limiting loads on anchors and bending stresses in wall to the design levels)
- Identification of the critical failure mode including sliding, overturning, base heave, anchor pullout, over stress in wall, and internal and external stability
The factor of safety for global stability is defined as the ratio of the available shear strength to the minimum shear strength needed for equilibrium. The factor of safety obtained by the FEM model is computed using a strength reduction procedure. In Plaxis, this procedure is specifically termed the phi-c reduction method, where phi and c refer to the friction angle and cohesion intercept, respectively. The underlying concept in phi-c reduction is that the soil strength is gradually reduced using a strength reduction factor. In this method, failure is considered to have occurred when a small reduction in strength leads to large changes in strains. The strength reduction factor corresponding to this state is considered to be the factor of safety on soil strength. The factor of safety computed using Plaxis gives results similar to conventional slip-circle analyses (Griffiths and Lane, 1999) where the factor of safety is the ratio of the true strength to the minimum strength required for equilibrium on a postulated slip surface.

A comparison of limit equilibrium and FEM stability analysis results for Wall 39-S1 on the project is shown in Figures 6 and 7. As previously mentioned, reducing the available strength in the FEM model by a value equal to the factor of safety results in increased strains in the soil mass. Eventually the strains start to accumulate along critical failure surfaces and any attempt to further reduce the strength results in increased rates of deformation. Figure 6 shows the strains resulting from any small change in soil strength near a factor of safety of 1.74. The red zones represent areas of high strain, and the blue zones represent areas of low strain. The result from a limit equilibrium analysis generated a factor of safety of 1.72 (Figure 7). The advantage of Plaxis is that it will find the most critical failure geometry without restrictions to a predefined shape of failure surface.

FIELD MONITORING
Prior to start of construction, geotechnical instruments were installed on the track rails to measure settlement and horizontal displacements. In addition, manual surveys were performed along the track rails to document settlement. Instruments included automated motorized total stations (AMTS) with reflective prismatic targets (Figure 8) located on the rails and manual survey points. Many of the instruments are designed to be read remotely and automatically. The settlement and displacement data from the remotely operated AMTS and reflective prismatic target system is transmitted via radio/cell phone to a centralized data base and data management system (Figure 9) resulting in real-time web-based monitoring with automatic alerts in the event pre-assigned limits on track performance are exceeded. Example settlement data is shown in Figure 10.

DATA REVIEW
Originally, settlement data from eight cases were reviewed and analyzed where adjacent support of excavation was constructed. Of the eight cases, six were selected as case histories for this paper as shown in Table 4:
The remaining two cases were not included because the data showed unexplained discrepancy between measured and predicted settlements at discrete measurement locations. There is a possibility that these discrepancies could be due to unanticipated construction activity/ loading or measurement errors, which at present, could not be confirmed. The six selected case histories represent excavations for a wide range of construction including electrical duct banks, tunnel boring machine jacking and receiving pits, for construction of retaining walls, and for extensions to existing bridge abutments. The subsurface conditions included varying thickness of fills overlying fine-grained silt deposits over glacial outwash and glacial till soils. Depth to groundwater typically ranged from 11 ft. to 28 ft.

The settlement data was typically taken from the track rail located nearest to the support of excavation. The measured settlement data obtained along the track at the critical support of excavation section, used in the support of excavation design, was used in these analyses. Further information for each Case is shown in Table 5:

Table 4 - Summary of Support of Excavation Cases

<table>
<thead>
<tr>
<th>Case No.</th>
<th>Construction</th>
<th>Subsurface Information</th>
<th>Groundwater depth (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Electrical duct bank</td>
<td>22 ft of silty sand underlain by 14 ft of clayey silt followed by glacial deposits</td>
<td>16</td>
</tr>
<tr>
<td>2</td>
<td>Tunnel boring machine reception pit and open cut trench for track approach</td>
<td>Fill and glacial deposits</td>
<td>28</td>
</tr>
<tr>
<td>3</td>
<td>Tunnel boring machine reception pit and open cut trench for track approach</td>
<td>Fill and glacial deposits</td>
<td>28</td>
</tr>
<tr>
<td>4</td>
<td>Modular block wall</td>
<td>10 ft of miscellaneous fill underlain by glacial deposits</td>
<td>19</td>
</tr>
<tr>
<td>5</td>
<td>Utility jacking pits</td>
<td>8 ft of fill underlain by glacial deposits</td>
<td>20</td>
</tr>
<tr>
<td>6</td>
<td>Extension of existing bridge abutment</td>
<td>18 ft of miscellaneous fill underlain by glacial deposits</td>
<td>11</td>
</tr>
</tbody>
</table>
The six cases consisted of different support of excavation designs including cantilevered and braced systems (single and multiple, internally and externally), steel sheet piling, soldier piles and wood lagging, and other variations on these designs. The depth of excavation ranged approximately from 9 ft. to 28 ft. The distance from the support of excavation to the nearest rail ranged from approximately 2.5 ft. to 21 ft. As shown in Table 5, maximum measured settlement of the rails at the critical design sections ranged from 0.2 in. to 0.8 in. for Cases 1 through 5, and 3.6 in. for Case 6. The predicted settlements ranged from 0.25 in. to 1.02 in. Figure 11 is a plot of measured versus predicted settlement at the critical design sections. In Figure 11, the plot indicates four cases that fall nearly on the “Equal Value” line and 2 cases that plot above the line which indicate greater measured settlements than those predicted. These latter two cases are Case 1 (soldier pile and lagging, externally braced with deadman) and Case 6 (cantilevered steel sheet piling and soldier piles). Case 1 required pre-stressing of the deadman tie-rods which could have accounted for displacement of the deadman to develop passive resistance. For Case 6, the numerical modeling was performed using plane-strain conditions which assume an average stiffness of the composite cantilevered section. The maximum measured settlement may have occurred at a point between the steel soldier piles where the stiffness was less, potentially causing greater horizontal displacement of the support of excavation and ensuing increased settlement of the track.

Figure 12 is a plot of normalized measured settlement versus normalized distance from the support of excavation to the nearest rail (both normalized by excavation depth). As
seen in the plot, five cases (Cases 1 through 5) plot in Zone I (after Clough and O’Rourke, 1990; and Peck, 1969) for temporary braced sheet pile and soldier pile walls installed in sands or soft to hard clay (average workmanship). Case 6 plots in Zone II.

CONCLUSIONS
Numerical modeling using finite element solutions like Plaxis to predict the deformation and stability performance of support of excavation systems and adjacent railroad track alignments proved successful on the East Side Access project. Considering the difficult soil conditions, shallow groundwater, and other factors, predicting the performance of the railroad tracks was challenging, particularly considering the tightness of the deformation performance criteria imposed by the project. In five of the six cases presented, settlements predicted using FEM were in good agreement with measured values. In addition, for the six case histories presented in this paper, the measured settlements were within the range of settlements anticipated using industry standards.

Figure 1 – Long Island Railroad East Side Access Project
Figure 2 – Harold Interlocking and Sunnyside Rail Yard, Long Island City

Figure 3 – Project Site Development Conditions
Figure 4 – Finite Element Model of Site Conditions

Figure 5 – Model of Final Excavated Condition with Support of Excavation
Figure 6 – Stability Analysis – Finite Element Method (FS = 1.74)

Figure 7 – Stability Analysis – Limit Equilibrium Method (FS = 1.72)
Figure 8 – AMTS/Reflective Prism Survey System

Figure 9 – Data Acquisition and Management System
Figure 10 – Example Track Rail Settlement Data

Figure 11- Measured vs. Predicted Track Settlements
Figure 12 – Settlement vs. Distance from Track (normalized)

References


List of Tables and Figures

Table 1 - Engineering Design Parameters for Soil and Rock
Table 2 - Allowable Deformations for Railroad Tracks
Table 3 - Minimum Geostructural Factors of Safety
Table 4 - Summary of Support of Excavation Cases

Figure 1 – Long Island Railroad East Side Access Project
Figure 2 – Harold Interlocking and Sunnyside Rail Yard, Long Island City
Figure 3 – Engineering Design for Support of Excavation
Figure 4 – Project Site Development Conditions
Figure 5 – Finite Element Model of Site Conditions
Figure 6 – Model of Final Excavated Condition with Support of Excavation
Figure 7 – Stability Analysis – Finite Element Method (FS = 1.74)
Figure 8 – Stability Analysis – Limit Equilibrium Method (FS = 1.72)
Figure 9 – AMTS/Reflective Prism Survey System
Figure 10 – Data Acquisition and Management System
Figure 11 – Example Track Rail Settlement Data
Figure 12 – Measured vs. Predicted Track Settlements
Figure 13 – Settlement vs. Distance from Track (normalized)
Measured and Predicted Rail Performance Adjacent to Deep Utility/Structure Excavations

Anand V. Govindasamy, PhD, PE
Geocomp Corporation, Acton, MA
Martin Hawkes, PE
Geocomp Corporation, Acton, MA
Scott R. Bamford, PE
Geotechnical Consultant, Boston, MA
Long Island Rail Road East Side Access Project

Planned Project Improvements
- New track alignments
- Existing track realignments
- Utility installations
- Below-grade structures
- Bridge foundation extensions
- Track approach structures
- Permanent retaining walls

Design Challenges
- Deep excavations
- Active rail traffic
- Small SOE displacement tolerances
- Difficult soil conditions (loose fills/soft organics, dense glacial soils)
- High live loads (railroad dynamic loading)
- Shallow groundwater
- Utility installations under existing railroad alignments

Typical Subsurface Conditions
- Miscellaneous fill
- Organic soils
- Glacial till/Glacial outwash
- Marine clay deposits
- Bedrock

Support of Excavation Designs
- Cantilevered steel sheet piling
- Cantilevered soldier pile & lagging
- Externally/externally braced steel sheet piling
- Externally/externally braced soldier pile & lagging
- Up to 30 ft deep excavations
- Minimum 2.5 ft rail to excavation distance
Settlements

Stability Analysis - Incremental Strength Reduction

Factor of Safety - Incremental Strain

Stability Analysis - Method of Slices

Field Displacement Measurements
- Instrument surveys
- Remote monitoring

Remote AMTS Survey System
Data Acquisition and Management System

Settlement Points

Settlement Data

Data Analysis

Measured vs. Predicted Settlements

Settlement vs. Distance from Track (normalized)
Conclusions

• Good to reasonable agreement between measured and predicted track settlements
• Measured settlements within industry standards
• Numerical modeling by finite elements successful
• Finite element modeling can reliably ensure the safety of track performance

Acknowledgements

• Tutor-Perini
  Construction Manager
• Metropolitan Transportation Authority
  Owner
• General Engineering Consultant
  • Parsons Brinckerhoff
  • STV Inc.
  • Parsons Transportation
• Amtrak
  Owner

Questions