Foundation Design and Construction of North America’s Tallest Ferris Wheel

Tripathi, H.
Assistant Project Manager
e-mail: htripathi@langan.com

Boyer, R.D.
Senior Associate
e-mail: rboyer@langan.com

Langan Engineering & Environmental Services, Inc., NJ, USA

ABSTRACT
This paper presents foundation design and construction of North America’s tallest Ferris Wheel at Meadowlands Xanadu, a $2 billion proposed family entertainment development in East Rutherford, New Jersey, USA. The foundations for the Ferris Wheel were designed to consist of 1.2 m-diameter, 27 m long drilled shafts socketted 3.5 to 4.8 m in shale bedrock. The drilled shafts were designed with an individual compression capacity of 8 MN and a lateral load capacity of 245 kN. The paper will discuss the site subsurface conditions; geotechnical design of drilled shafts; lateral load capacity analysis; and Osterberg Cell (O-cell) testing. The O-cell load test confirmed the shaft design parameters and demonstrated the potential for higher rock side shear capacities and significant end bearing.

1. INTRODUCTION
This paper presents the foundation design and construction of North America’s tallest (87 m) Ferris Wheel at Meadowlands Xanadu, a $2 billion proposed family entertainment development in East Rutherford, New Jersey, USA. The Ferris Wheel will rise over the Meadowlands and will be an iconic structure offering sweeping vistas of the New York City skyline and the Hudson River through its glass-enclosed, climate controlled capsules. The Ferris Wheel foundation caps and drilled shaft layout plan were designed by a Boston-based Structural Engineering firm, McNamara/ Salvia. Langan Engineering & Environmental Services, Inc. (Langan) was the Geotechnical consultant and Underpinning & foundation Skanska was the foundation Contractor. The paper will discuss the site subsurface conditions; geotechnical design of drilled shafts; lateral load capacity analysis; and Osterberg Cell (O-cell) testing.

2. GEOLOGY AND SITE HISTORY
Regional Geology
The existing Meadowlands Xanadu site is located within the Hackensack River basin of the Piedmont physiographic province of the Appalachian Highlands. The underlying bedrock in the region is characterized as reddish-brown silty to sandy mudstone and siltstone belonging to the Brunswick Formation dating from the Lower Jurassic and Upper Triassic Period. The bedrock directly beneath the project site is identified as red shale and is encountered at depths of about 8 to 30 meters below existing grade. In more recent geologic times, the site vicinity was covered by glacial advances that scoured the Hackensack River Valley resulting in glacial till deposits over most of the bedrock in the area. Glacial meltwaters flowing into the lake deposited alternating layers of sand, silt, and clay at the lake bottom resulting in the formation of “varved clays.” The rising ocean levels brought about organic deposits (peat or “meadowmat,” and organic silt/clay) as tidal marsh areas developed in the Hackensack River basin.

Site History
The site is composed of a 1.2 to 5 meter thick deposit of miscellaneous fill material consisting of sand, silt, organic silt, peat, and other miscellaneous debris. Approximately 3 million cubic meters of dredged materials were needed to complete the construction of the Sports Complex. Because of the challenging site and geologic conditions, the structures at the site were supported on pile foundations, with ground improvement consisting of surcharging to reduce post-construction settlement in other areas of the site.

3. SUBSURFACE INVESTIGATION
An extensive subsurface investigation program consisting of drilling over 200 borings, performing several cone
penetration tests (CPTs), and laboratory tests for consolidation, shear strength, and index properties were performed for the Meadowlands Xanadu development. Additionally, five borings were drilled within the proposed Ferris Wheel foundation footprint. The borings were inspected by Langan and drilled to depths ranging from 27 to 30 meters below existing grade and Standard Penetration Test (SPT) was performed as part of the sampling procedure. Rock coring (3 to 4.5 m) was performed in all borings using an NX core barrel to assist in determining the depth, type, and quality of the underlying rock.

Subsurface Conditions
The subsurface conditions encountered in the borings drilled for the Ferris Wheel generally consisted of 4.3 to 5.5 m of loose to very dense fill underlain by 0.5 to 0.6 m of soft to stiff organics, 9 to 13 m of soft to very stiff varved clay, 2.6 to 5.5 m of dense to very dense glacial till, and shale bedrock. The upper 0.7 to 2.4 m of shale rock was found to be weathered. The depth to rock ranged from 21 to 23 m below existing grades. The rock core recoveries and rock quality designations averaged 95% and 67%, respectively. Groundwater was encountered in all the borings at depths ranging from 1.8 to 2.7 m below existing grades.

3. FOUNDATION DESIGN

Structural Loads
The proposed Ferris Wheel amusement ride is to consist of an approximate 67 meter-diameter wheel mounted on supports. Wind tunnel testing was performed by Rowan Williams Davies & Irwin, Inc. to model the loading on the Ferris Wheel. The wind tunnel testing predicted significant reduction in structural loads compared to the conservative loads derived in accordance with the International Building Code (IBC). The project Structural Engineer, McNamaara/Salvia, determined the maximum compression and lateral load on drilled shafts of 8 MN and 245 kN, respectively.

Drilled Shaft Design
Because of these high axial, uplift, and lateral loads associated with the Ferris Wheel structure, use of a deep foundation system consisting of drilled shafts was designed and recommended by Langan. The drilled shaft for the Ferris Wheel was designed to achieve a design compression capacity of 8 MN, uplift capacity of 2.7 MN, and a lateral load capacity of 245 kN. The drilled shaft was designed assuming that the entire capacity will be derived from the peripheral shear strength between the rock and concrete along the length of the rock socket. An allowable bond stress between rock and concrete of 690 kPa for shale was used for design. For uplift capacity, an allowable bond stress of 228 kPa was used for design. The allowable stresses for concrete and steel were based on the 2006 International Building Code (IBC) – concrete: 33% of the compressive strength of concrete; steel casing: 35%, of the yield strength of steel; and for reinforcement steel: 50% of the yield strength of steel. The drilled shaft design was checked for allowable structural capacity, geotechnical capacity, steel elongation, and lateral load capacity.

The lateral load capacity of the drilled shaft was the limiting criteria in selecting the diameter of the shaft. The lateral load capacity and deflection analysis was performed for 0.9 m and 1.2 m diameter shafts using the LPILE program. The group effect was considered by reducing the subgrade modulus for each soil layer by a reduction factor of 0.25 (NAVFAC DM 7.2); Table 1 provides the soil properties used for analysis. For sands the p-y curves were selected by LPILE as per Reese et al. 1974. For stiff clays (Reese et al. 1975) in the presence of free water and soft clays (Matlock 1970), the p-y curves were obtained by correlations with the laboratory compressive strength test results and consistency of clays. The soil strain parameters (at 50% strain) for stiff and soft clays were assumed to be 0.007 and 0.02, respectively.

Table 1: Soil Properties for Lateral Capacity Analysis

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Unit Weight (kN/m³)</th>
<th>Friction Angle (deg.)</th>
<th>Shear Str. (*) (kPa)</th>
<th>Subgrade Modulus (MPa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill</td>
<td>18.9</td>
<td>32</td>
<td>-</td>
<td>6.1</td>
</tr>
<tr>
<td>Sand</td>
<td>9.0</td>
<td>32</td>
<td>-</td>
<td>4.1</td>
</tr>
<tr>
<td>Peat</td>
<td>2.7</td>
<td>-</td>
<td>7.2</td>
<td>-</td>
</tr>
<tr>
<td>Stiff Clay</td>
<td>9.0</td>
<td>-</td>
<td>81.4</td>
<td>33.9</td>
</tr>
<tr>
<td>Soft Clay</td>
<td>7.5</td>
<td>-</td>
<td>16.8</td>
<td>-</td>
</tr>
<tr>
<td>Sand</td>
<td>9.0</td>
<td>36</td>
<td>-</td>
<td>15.2</td>
</tr>
<tr>
<td>Glacial Till</td>
<td>10.6</td>
<td>18</td>
<td>95.8</td>
<td>33.9</td>
</tr>
</tbody>
</table>

*Based on laboratory test data from unconsolidated undrained compression tests.

The LPILE analysis was based on a worst-case and average soil profile analyzing free-head and fixed-head conditions at the top of the shaft for a 1.2-m-diameter shaft; see Figure 1. The allowable lateral load capacity of the shaft was selected as 245 kN based on a 9.5 mm free-head deflection of the shaft head.
The drilled shafts were designed to consist of 1.2-m-diameter permanent steel casing (minimum yield strength of 345 MPa with an allowance for 1.58 mm corrosion loss), a 3.5-m-long, 1-m-diameter rock socket embedded in competent shale with a minimum center-to-center spacing of 3 m. The shaft reinforcement was designed to consist of a Contractor proposed HP360x132 core beam with 22.2-mm-diameter welded shear studs as opposed to a conventional rebar cage. In addition to the centralized core beam, the top 2.4 m of the shaft also included a 24#36 rebar cage.

4. FOUNDATION CONSTRUCTION

Drilled Shaft Construction Methods

A boring was drilled at the location of the test shaft to confirm the depth and quality of competent rock prior to installing a test shaft. The test shaft and all subsequent production shafts were installed by the foundation Contractor, Underpinning & Foundation Skanska, Inc. using Bauer BG24 and BG28 drill rigs. The 1.2-m-diameter steel casing was vibrated at least 0.45 m into weathered shale rock using an ICE 6680 hydraulic vibratory hammer. Pre-augering through the fill was required prior to vibrating the permanent steel casing at several shaft locations. The soil within the casing was removed using a 1-m-diameter pilot auger. Rock socket installation was performed using a 1-m-diameter rock auger. The bottom of the shaft was thoroughly cleaned of any soil and rock using a 1-m-diameter clean-out bucket. Quality control procedures consisted of rock socket video inspection immediately prior to placement of reinforcement within the shaft. An HP360x132 core beam attached with a 200-mm-diameter tremie pipe was placed at the center of the shaft after the inspection of the shaft and prior to concreting the shaft. Because of the construction schedule, the test shaft was concreted about 20 hours after the socket was excavated. Shafts drilled in shale and left open for some time may oxidize and then become soft from adsorbing water from concrete (Osterberg 2000).

Osterberg Cell (O-Cell) Test

The Osterberg Cell (O-cell) is a hydraulically driven, high capacity, sacrificial loading device installed within the foundation system to be tested. Utilized in a shaft, the O-cell has the capacity to apply loading in two directions, upward against side-shear and downward against end-bearing. The O-cell derives all reaction from the soil and/or rock system. The O-cell load test was conducted by LOADTEST, Inc. 7 days after the shaft concrete reached the minimum design compressive strength. The test shaft consisted of one 0.53-m-diameter O-cell located 0.72 m above the tip of the shaft (at el-26.2m). O-cell instrumentation included three Linear Vibrating Wire Displacement Transducers (LVDTs) positioned between the upper and lower plates of the O-cell. Two telltale casings were attached to the reinforcing cage, diametrically opposed from one another, extending from the top of the O-cell assembly to beyond the top of concrete. Six levels of two sister bar vibrating wire strain gauges (SG#1 to SG#6) were installed to assess the side shear load transfer. Two steel tubes were also installed within the shaft to provide access for post-test grouting of the annular void surrounding the O-cell assembly.

During the load test, shaft compression was monitored and recorded by LVDTs and automated digital survey levels. A Bourdon pressure gauge and a vibrating wire pressure transducer were used to measure the pressure applied to the O-cell. The shaft was loaded in 15 increments to a bi-directional gross O-cell load of 8.48 MN and unloaded in four equal decrements. The loading was performed in accordance with the Quick Load Test Method for Individual Piles (ASTM D1143), holding each successive load increment for eight minutes.

O-Cell Load Test Results

Theoretically, the O-cell does not impose an additional upward load until its expansion force exceeds the buoyant unit weight of the shaft above the O-cell. The maximum upward applied net load to the upper side shear was 8 MN, with a total upward movement of the O-cell of 1.25 mm; see Figure 2.

At the maximum O-cell load of 8.48 MN, the average downward O-cell movement was 2.11 mm. The load taken in shear by 0.72 meters of shaft section beneath the O-cell was calculated to be 2.4 MN using an extrapolated unit side shear value of 1,221 kPa. The unit end bearing at the base of the shaft was calculated to be 6,210 kPa at the above noted displacement. Based on the strain gauge data and estimated shaft stiffness, the side shear resistance of the test shaft was calculated; see net unit shear values between the strain gauges in Table 2.

The average rock socket unit side shear was found to be 811 kPa and was greater than the design rock bond stress of 690 kPa. Table 2 shows that the load transfer through
the shaft is much more pronounced within the rock socket. The test shaft mobilized a combined end bearing and side shear resistance of 16.5 MN during the test. For an equivalent top loading of 8 MN and 16 MN, the adjusted test data indicates that this shaft would settle approximately 4.8 mm and 10.2 mm, respectively, including 4.4 mm and 8.9 mm of elastic compression. The combined end bearing and lower side shear creep data indicate that no apparent creep limit (Housel 1959) was reached at a movement of 2.11 mm.

Table 2: Average Net Unit Side Shear Values at Max. Load

<table>
<thead>
<tr>
<th>Load transfer zone</th>
<th>Elev. (m)</th>
<th>Disp (mm)</th>
<th>Net unit side shear (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SG#6 to No Shear</td>
<td>–8.4 to 0</td>
<td>0.74</td>
<td>0</td>
</tr>
<tr>
<td>SG#5 to SG#6</td>
<td>–19.9 to – 8.4</td>
<td>0.76</td>
<td>0</td>
</tr>
<tr>
<td>SG#4 to SG#5</td>
<td>–21.4 to –19.9</td>
<td>0.78</td>
<td>8</td>
</tr>
<tr>
<td>SG#3 to SG#4</td>
<td>–22.6 to –21.4</td>
<td>0.81</td>
<td>67</td>
</tr>
<tr>
<td>SG#2 to SG#3</td>
<td>–23.3 to –22.6</td>
<td>0.81</td>
<td>408</td>
</tr>
<tr>
<td>SG#1 to SG#2</td>
<td>–24.0 to –23.3</td>
<td>0.86</td>
<td>755</td>
</tr>
<tr>
<td>O-cell to SG#1</td>
<td>–25.5 to –24.0</td>
<td>1.07</td>
<td>1,017</td>
</tr>
<tr>
<td>Average rock socket</td>
<td>–25.5 to –2.6</td>
<td>1.02</td>
<td>811</td>
</tr>
</tbody>
</table>

*Average displacement of load transfer zone.

Since the test shaft was to be used as a production shaft, the Contractor filled the O-cell and annular void in the shaft (as a result of O-cell expansion) with grout. Forty-three production shafts were installed subsequent to the successful O-cell load test and the foundation construction was completed on schedule.

5. CONCLUSIONS

For shafts with high lateral load capacities, the diameter of the shaft becomes a limiting design factor. The Ferris Wheel foundation construction schedule utilized the O-cell load test to successfully confirm the shaft design parameters. Average rock bond stress was estimated to be 811 kPa. The O-cell load test data indicated that for an equivalent top loading of 8 MN, the shaft would settle approximately 4.8 mm. The combined end bearing and lower side shear creep data indicate that no apparent creep limit was reached. The test shaft concreted 20 hours after socket installation and the presence of water in the shaft did not have a detrimental affect on the rock bond stress. The completed O-cell test demonstrated the potential for higher rock side shear capacities and significant end bearing. For larger scale projects, the O-cell should be utilized to refine the design assumptions to justify higher allowable side shear resistance and end bearing and provide significant cost savings in foundation construction.

ACKNOWLEDGMENTS

The authors would like to acknowledge the following Langan personnel: Lijian Zhou, Matt Meyer, Arthur Roesler, Nick Starzynski, and Ronald Manney.

Copyright Permissions

For any material which is not original, copyright permission to reproduce the material must be obtained in advance in writing by the authors.

REFERENCES


