Prediction of behavior of structures due to large bore EPBM tunneling at the Port of Miami

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ABSTRACT: The Port of Miami Tunnel Project (POMT) consists of twin Earth Pressure Balance Tunnel Boring Machine (EPBM) tunnels excavated within complex mixed face conditions beneath existing surface structures located on Dodge Island, Miami, Florida. Tunneling occurs beneath an existing Seawall structure, passenger receiving facilities and buildings. The structures range in age up to 40 years old and exhibit various levels of deterioration due to the corrosive environmental conditions local to Miami. A potential damage assessment was performed in a two-stage approach to first identify higher risk structures and then evaluate the highlighted structures with the use of a three-dimensional finite element analysis, which was calibrated using settlement data obtained from the beginning of the EPBM drive. During tunneling, monitoring data was provided daily and compared to results from the finite element model to observe the overall performance of the structure and to identify any trends which may result in potentially damaging structural displacements. After construction, final observed structural displacements were compared to the numerical modeling predictions to assess the validity of geotechnical parameters and tunneling performance.

1 Introduction

Currently, the only access to the Port of Miami for shipping traffic involves navigating busy downtown city streets, causing traffic congestion and limiting economic development of the northern portion of Miami’s Central Business District. To alleviate these issues, the Port of Miami Tunnel is under construction, which will provide a direct underground connection from the Port of Miami at Dodge Island via Watson Island to I-395 and all other highways. To complete the project, a Public-Private Partnership (PPP) was established between the Florida Department of Transportation, Miami-Dade County, the City of Miami, Meridiam Infrastructure Finance, and Bouygues Travaux Publics as part of the design-build-finance-operate-and-maintain (DBFOM) contract. Bouygues Civil Works Florida (BCWF) acted as the prime contractor for the project.

The project consists of twin bored tunnels from Watson Island to Dodge Island, passing beneath a number of existing structures on Dodge Island. Several of the structures are currently in use by the cruise lines – which contribute greatly to the local economy – and were not permitted to be negatively impacted by the tunneling operations. The critical structures identified for the potential damage assessment are shown in Figure 1 and listed below:

- Seawall & Bulkhead – Corrugated steel sheet piling seawall, reinforced concrete pile cap, and tieback to sheet piling “dead man” anchor system. Originally constructed in the late 1950’s and exhibits corrosion and deterioration of concrete and steel due to seawater exposure.
- Pedestrian Bridge – Elevated steel structure providing access to loading gantries for cruise ship embarking. Supported by reinforced concrete columns on drilled pile foundations.
- Shed #2 – Open air reinforced concrete frame structure currently used for cruise ship supply storage. Originally constructed in 1967, the building shows extensive superficial concrete deterioration and spalling due to environmental exposure.
Seaman’s Center Swimming Pool – Recreational swimming pool within the influence zone. The owner was concerned with potential concrete cracking and water loss due to ground settlements.

Figure 1. Existing Dodge Island structures near tunnel alignments

2 Geological Setting & Ground Parameters

The geologic history of the Miami area provided challenging geotechnical conditions with respect to tunneling. A specially designed Earth Pressure Balance Tunnel Boring Machine (EPBM) was selected for the project which allowed for tunneling beneath the water table and channel through difficult mixed face conditions. Tunneling generally occurred through four geologic formations:

- Fort Thompson Formation
- Anastasia Formation
- Key Largo Formation
- Tamiami Formation

The depositional history of the rock units found on Dodge Island begins at the end of the Pliocene Epoch—roughly 2.59 million years ago—when a shallow, sub-tropical sea covered much of the land that is now Florida. The conditions in this environment favored the growth and expansion of a plethora of carbonate-producing organisms, including corals and bryozoans, marine plants and algae, whose remains accumulated to produce the sediments that would eventually become the Tamiami Formation. Siliciclastic sediments from the eroding Appalachian Mountains to the north were carried into the area by rivers, streams and longshore currents and deposited on the broad Florida Platform, later to become the layers and lenses of sands, clays and silts found within the Tamiami unit. The beginning of the Pleistocene Epoch, and, subsequently, the Quaternary Period, saw the arrival of the Ice Ages roughly 1.8 million years ago. During this time in geologic history, fluctuations of several hundred feet above and below present sea level were caused by the repeated growth and melting of the great glaciers covering much of northern North America, Europe and Siberia. The expansive coral reefs covering the Florida Platform at this time maintained their foothold along the edge of the Florida Platform, migrating upwards during rising sea levels and retreating during interglacial periods. In this manner during successive phases of growth, the remains of corals and bryozoans, invertebrate shells, marine plant and algal debris all accumulated to form the Key Largo Limestone, the Anastasia and Fort Thompson formations, and the Miami Limestone present on Dodge Island.

The geotechnical investigation program conducted for the Port of Miami Tunnel Project identified eight (8) distinct ground strata in the area, with most of the tunneling being within Strata 6 through 8. Dodge Island is a man-made island formed of reclaimed land which was dredged from Biscayne Bay and deposited during the deepening of the Port of Miami. These upper fill layers generally consist of sand,
silty sand, and silt and overlay the rock formations. Beneath, the Fort Thompson Formation is characterized by a pale orange to yellowish-grey fossil-rich wackestone/packstone containing corals, bryozoans and mollusks (Stratum 6). Underlying the Fort Thompson Formation is the porous, coquina and coquinooid limestone of the Anastasia Formation (a grainstone) and the fossil-rich, coralline Key Largo Limestone (a boundstone), which contains coral heads, bryozoans and mollusks encased in calcarenite (Stratum 7). This unit also contains isolated zones of loose, uncemented sands and silts much weaker than the surrounding limestone. The Anastasia and Key Largo formations may occur as interfingered lenses and layers within the basal Fort Thompson Formation. The ground investigation terminated in the Tamiami Formation (Stratum 8), a grey, porous grainstone with layers and lenses of shelly sands and sands interbedded with clays and silts.

While the Fort Thompson and Tamiami Formation generally exhibit high degrees of cementation resulting in a strong competent limestone, the interbedded Anastasia and Key Largo Formations present grooves of uncemented sands and silts between the more competent rock material. Consequently, geotechnical parameters derived during the ground investigation of this stratum varied widely depending on whether the borehole penetrated the rock or the uncemented sands/silts. Modeling of this material proved to be difficult and predictions of surface settlements from the finite element analysis were highly dependent on the geotechnical parameters chosen for this stratum; however, due to the large variation of observed data, conservative estimations for compressive strength and Elastic Modulus were necessitated.

Figure 2. Samples of Stratum 7 material – Key Largo Limestone

3 Analysis Approach

3.1 Background

A thorough literature review of previous research on building damages resulting from tunneling-induced ground movements was performed and is briefly summarized in Table 1. Recent research on building damage assessments was also reviewed but is essentially based on the work of the authors included in Table 1. A two-stage process for assessing the damage caused to buildings by excavation-induced ground movements as proposed by Mair et al (1996) was selected for the project. An increasing level of rigor is applied at each stage of the assessment. This approach has been successfully adopted on various major tunneling projects throughout the world, particularly in London, including the Jubilee Line Extension, the Channel Tunnel Rail Link and most recently Crossrail.

Stage 1 involved the calculation of ground surface movements resulting from tunneling. Excavation-induced ground surface movements can be predicted with a reasonable degree of confidence for a ‘greenfield’ site. In practice however, the greenfield site assumption tends to be conservative as existing building foundations and underground structures will interact with the settlement trough to modify its shape and typically decrease its overall magnitude. Therefore, Stage 1 was intended to act as a conservative filter to identify structures that will require a more detailed Stage 2 assessment.
Table 1. Literature reviewed for assessment

<table>
<thead>
<tr>
<th>Author &amp; Year</th>
<th>Title</th>
<th>Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>Skempton &amp; MacDonald (1956)</td>
<td>The Allowable Settlements of Buildings</td>
<td>Provides observed building damages correlated with angular distortion limits based on numerous infilled steel, reinforced concrete framed, and load bearing wall buildings.</td>
</tr>
<tr>
<td>Attewell &amp; Woodman (1982)</td>
<td>Predicting the dynamics of ground settlement and its derivatives caused by tunnelling in soil</td>
<td>Overview of case histories where longitudinal ground settlements ahead of tunneling face have been recorded. Empirical formula suggested for predicting longitudinal ground settlements.</td>
</tr>
<tr>
<td>Boscardin &amp; Cording (1989)</td>
<td>Building Response to Excavation-Induced Settlement</td>
<td>Suggest deep beam model for existing surface masonry structures. Provides categorized damage levels with typical visible damages and building tensile strain limits.</td>
</tr>
<tr>
<td>Mair, Taylor, &amp; Burland (1996)</td>
<td>Prediction of ground movements and assessment of risk of building damage due to bored tunnelling</td>
<td>Suggest two-staged approach for evaluating potentially damaging ground settlements due to bored tunneling. Recommends simplified greenfield analysis to calculate ground strains to highlight high risk structures. High risk structures should be evaluated in a secondary stage considering soil-structure interaction effects.</td>
</tr>
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Two greenfield ground settlement prediction methods were compared – a Gaussian Error function proposed by O’Reilly & New (1982) and a two-dimensional finite element analysis provided by BCWF. The two methods provided similar maximum vertical settlement predictions; however, a relatively wider settlement trough was predicted with the numerical analysis. Additionally, the influence of ground settlements developing longitudinally as the EPBM’s passed by the structures was evaluated with an empirical cumulative error function proposed by Attewell & Woodman (1982). Resulting ground strains and angular distortions were calculated and compared to damage criteria developed for each structure. If the structure was determined to fall into a damage category higher than Slight, it was advanced to Stage 2.

Structures which progressed to the Stage 2 analysis were evaluated by a calibrated three-dimensional ground-structure interaction finite element model. Displacements, strains, and loadings were then explicitly compared to the structural element capacities to discern the anticipated performance of the structures.

3.2 Damage Criteria

Damage criteria were developed based on suggestions by Boscardin and Cording (1989) for limiting tensile strains and differential deflection limits suggested by applicable design codes – specifically ACI 318 for concrete structures and AISC Design Guide 3 for steel structures. Design criteria specified extremely strict limits for building settlements and strains; therefore, suggested values for building strain limits proposed by Boscardin and Cording (1989) were adjusted to account for the existing condition of the structures. The existing conditions of the structures were categorized utilizing classes and descriptions of typical damage levels proposed by Boscardin and Cording (1989) and used at the baseline damage category. Effectively, individual damage criteria were developed to incorporate the deterioration already experienced by the older structures while less stringent criteria were used for the newer structures.
Table 2. Damage criteria for existing Dodge Island structures

<table>
<thead>
<tr>
<th>Damage Category</th>
<th>Normal Degree of Severity</th>
<th>Seawall &amp; Bulkhead</th>
<th>Pedestrian Bridge</th>
<th>Shed #2</th>
<th>Swimming Pool</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Limiting Tensile Strain (%)</td>
<td>Allowable Angular Distortion (°/l)</td>
<td>Limiting Tensile Strain (%)</td>
<td>Allowable Angular Distortion (°/l)</td>
</tr>
<tr>
<td>0</td>
<td>Negligible</td>
<td>N/A</td>
<td>0 to 0.05</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>1</td>
<td>Very Slight</td>
<td>N/A</td>
<td>&lt; 1/300</td>
<td>0.05 to 0.075</td>
<td>&lt; 1/300</td>
</tr>
<tr>
<td>2</td>
<td>Slight</td>
<td>0 to 0.075</td>
<td>0.075 to 0.15</td>
<td>0 to 0.075</td>
<td>0.025 to 0.1</td>
</tr>
<tr>
<td>3</td>
<td>Moderate Slight / Severe</td>
<td>0.075 to 0.225</td>
<td>&gt; 1/300</td>
<td>0.15 to 0.3</td>
<td>&gt; 1/300</td>
</tr>
<tr>
<td>4 / 5</td>
<td>Very Severe</td>
<td>&gt; 0.225</td>
<td>&gt; 0.3</td>
<td>&gt; 0.225</td>
<td>&gt; 0.25</td>
</tr>
</tbody>
</table>

* Angular distortion defined as differential vertical settlement of structural element support divided by length

### 3.3 Three-Dimensional Finite Element Analysis

Due to the age, tightened damage criteria limits, and highly three-dimensional behavior of the structures, the Seawall & Bulkhead and Shed #2 were advanced to Stage 2. In Stage 2, an independent three-dimensional ground structure interaction analysis, explicitly modeling the existing Dodge Island structures was performed. This allowed for calculation of increases in structural element loadings which were compared to the design capacity to determine whether the structure was still within acceptable loading limits.

A three-dimensional finite element analysis was performed for the area surrounding these structures using the Midas GTS 2012, v.1.1 software utilizing the DIANA solver by TNO DIANA BV for nonlinear analyses. Ground materials were modeled with three-dimensional 4-node tetrahedral elements with a Mohr-Coulomb failure criterion. The extents of the modeling geometry and various ground strata are shown in Figure 3.

Figure 3. Overall model geometry

Shed #2 is a reinforced concrete frame structure consisting of prefabricated double T roof beams resting on S-shaped support elements on drilled pile foundations (see Figure 4). The structure is nearly perpendicular to the tunnel alignment but does not provide structural connecting elements laterally between the frames besides the roof Double T’s. Therefore, the structure behaves “flexibly”
during the tunnel construction and is tolerant to differential displacements between the frames. In the numerical model, the structure was approximated as one-dimensional elastic beam elements and two-dimensional elastic plate elements (see Figure 5).

Horizontal displacements of the Seawall towards the EPBM face resulting in global stability issues of the Seawall and dead man system were of concern. Therefore, the Seawall, dead man, and tie backs were explicitly modeled with two-dimensional elastic plate elements and one-dimensional tension-only elastic rod elements (see Figure 5).

![Figure 4. Interior of Shed #2, showing S-shaped support frames and roof Double T’s](image)

To simulate the EPBM excavation, the Elastic Modulus of the material within the excavated volume along the alignment was reduced by a “Softening Factor”. This process models movement of ground material into the excavated area and into the face prior to installation of the tunnel lining and mobilizes the strength of the surrounding ground. While the softening factor is generally obtained from research or empirical relationships; in this case monitoring data was available for the Watson Island segment of the alignment and was used to calibrate the softening factor used in the model. A lower and upper bound Softening Factor of 30% and 50% were selected to incorporate the variety of ground settlements observed at Watson Island during the beginning of the EPBM drive. However, as the existing structures were explicitly modeled, it allowed for the softening factor to be gradually increased until loadings on certain structural elements reached their capacities (as specified by the relevant design codes) thus determining the upper bound limits for the allowable settlements.

3.4 Predicted vs. Measured Ground Settlements

Extensive monitoring installed on the existing structures provided the opportunity to observe the structural displacements during tunneling and compare to movements predicted by the finite element analysis. The Seawall & Bulkhead was monitored by mounted three-dimensional optical prisms surveyed by an automated electronic theodolite total station. Horizontal movements of the sheet pile
toe were observed with the use of multi-point borehole extensometers (MPBX’s) installed behind the seawall.

Mounted three-dimensional optical prisms were also installed throughout Shed #2 and the Passenger Bridge and were recorded twice per day by an automated total station when the EPBM was within the vicinity. Rotations of the S-frame supports were monitored by two-dimensional electronic tilt meters, while existing concrete cracks and expansion joints were monitored by several crack meters.

Figure 6. Instrumentation in Shed #2 (left), and automated total stations used for data collection (right)

The analysis predicted a relatively wide settlement trough due to the more competent rock material above the tunnel crown acting to spread the surface settlements laterally. This was confirmed by the instrumentation installed on Dodge Island and led to only minor differential displacements and rotations of the structures. Figure 7 presents the structural deformations predicted by the modeling and Figure 8 compares the predictions to the settlements observed after the Eastbound EPBM construction.

Figure 7. Vertical structural displacements predicted by 30% softening model of Shed #2

Figure 8. Comparison of predicted to observed structural settlements for Shed #2
As expected, settlements observed at the Seawall & Bulkhead structure were less than those observed at Shed #2 due to the stiffening effect of the steel sheet piling. Vertical settlements exhibited a wide, shallow trough resulting in minimal differential displacements as shown in Figure 9.

![Seawall Pile Cap Vertical Settlement](image)

**Figure 9.** Comparison of predicted to observed structural settlements for the Seawall & Bulkhead

## 4 Conclusions

While the settlements observed from instrumentation of the structures proved to be less in magnitude than those predicted by the analysis, the settlement troughs displayed the wide and shallow trough anticipated by the three-dimensional finite element model. This would suggest that the Softening Factor selected during the calibration exercise was relatively accurate; however, the geotechnical parameters chosen for Stratum 7 were conservative. Conservatively, lower bound parameters were selected for this material; however, it has been shown at the Port of Miami Tunnel Project that the composite structure of the interbedded Anastasia and Key Largo Formation can provide competent support for the overlying strata.

Potential of damage to the existing Dodge Island structures was successfully evaluated using the two-staged approach discussed in section 3.1. Performing the Stage 1 analysis followed by the more vigorous Stage 2 analysis for highlighted structures, less effort and resources were expended on structures which would not be at risk of damage. Due to the highly three-dimensional nature of the interaction between the tunneling-induced settlements and the movements of the structures, a three-dimensional numerical analysis was considered critical. By explicitly modeling the twin tunnels and the existing structures, the additional straining and loading caused by the underground operations on Shed #2 and the Seawall could be assessed. Additionally, the ultimate allowable deformations of the structures could be determined by adjusting softening factors until the structural elements reach their capacities. This provided verification that, even if observed ground settlements reached conservative predictions from the Stage 2 analysis, there would still exist additional reserve capacity beyond even what is stipulated by the design codes.

## 5 References


Midas GTS 2012 (v1.1) by MIDAS Information Technology Co., Ltd. Build: December 28, 2011. Solver: DIANA Solver by TNO DIANA BV.
