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PREDICTION AND PERFORMANCE OF DEEP EXCAVATIONS FOR COURTHOUSE STATION, BOSTON

by

Andrew J. Whittle¹ M.ASCE; Gonzalo Corral² A.M.ASCE; Lucy C. Jen³ M.ASCE; and Robert P. Rawnsley⁴ M.ASCE

ABSTRACT

Construction of the Silverline Courthouse Station in South Boston involved 18m deep excavations at a site underlain by more than 24m of normally and lightly overconsolidated Boston Blue Clay. The excavations were supported by 27m deep ‘floating’ diaphragm wall panels and 5 levels of pre-loaded cross-lot bracing. This paper compares the measured performance of the excavation support system with Class A finite element predictions prepared during the original design phase, and with results of Class C analyses using information obtained during construction. The numerical analyses used data from a special test program of laboratory and in situ tests (reported by Ladd et al, 1999) at a nearby site. The analyses represent coupled consolidation within the soil mass and anisotropic stress-strain-strength properties of BBC using the MIT-E3 soil model. Class A analyses generally overestimate the lateral wall deflections and underestimate the measured strut loads as preloading was not included in the original FE model. However, they provide remarkably consistent predictions of the measured soil deformations including settlements, lateral spreading and sub-grade basal heave. The Class C analyses refine the stratigraphic section, in situ pore pressures, construction timeframe and strut pre-loads using data available at the time of construction, but make nominal changes in soil properties. With these limited changes, the Class C model is able to achieve excellent agreement with the measured data.

KEYWORDS: Braced excavation, field measurements, finite element analyses, clay behavior, predictions vs measurements.

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INTRODUCTION

Completion of the I-90 Connector tunnels (part of the CA/T project, 1995-2003) and South Piers Transitway (a bus rapid transit system now known as Phase 2 of the Silver Line; 2000-2004) provide transportation links that have radically transformed the South Boston waterfront area, an area now referred to as the Innovation District; Figure 1. There has been an extensive program of geotechnical research at MIT associated with the design and performance of the excavation support systems for these tunnel sections. Whelan (1995) documented the measured performance of anchored walls for the open-cut, boat sections for the I-90 Connector Tunnels (CA/T Contract 4A; Fig. 1), while Jen (1998) carried out extensive finite element analyses of the project using an advanced effective stress soil model (MIT-E3; Whittle & Kavvadas, 1994; Whittle et al., 1994) to represent the behavior of the Boston Blue Clay. The model input parameters were selected using results from a program of laboratory and in situ tests performed at the South Boston Special Test site (SBST; Ladd et al., 1999) shown in Figure 1.

Jen (1998) also reported a series of Class A (i.e., a priori) predictions for the proposed design of cross-lot braced excavations for the Courthouse Station of the MBTA South Piers Transitway (contract section CC07; Fig. 1), also located less than 400m from the SBST site. This project involved 18m deep, 33m wide excavations for the station platforms in an area where deposits of normally to lightly overconsolidated Boston Blue Clay (BBC) extend to a depth of more than 40m. Conventional calculations showed a basal stability factor at the lower limit of accepted values. The temporary works design for this section (contract CC07) used a floating support system with four or five levels of cross-lot bracing, without ground improvement. In contrast, the designs for an adjacent contract (CC05; Fig. 1) used jet grout columns (up to 32m deep) to stabilize the clays below the base of the excavation (Hurley, 2004).

Figures 2a and 2b compare one of the original designs analyzed by Jen (1998) with the final (as built) support of excavation. We refer to Fig. 2a as the ‘original design’ considered in our Class A analyses. Based on these analyses, the design engineer actually specified a minimum of 5 levels of support with maximum horizontal and vertical spacing between struts of 6.7m and 3.3m, respectively. The contractor selected the final design system based on independent analyses. This is referred to as the ‘final design’, Fig. 2b, and uses 1.22m thick diaphragm wall panels (compared to 0.9 considered by Jen, 1998) with larger struts (higher stiffness and allowable capacity).

The actual excavation was closely monitored through an extensive program of field instrumentation with very close control of the excavation process to minimize risks associated with basal instability. The contract specifications also required the contractor to excavate the
final stage in a checkerboard pattern when preparing the installation of the base slab for the permanent works. The temporary works were completed during the period May 2000 – July 2001 and the tunnel was opened in 2004.

Corral (2013) has made extensive use of data from the Transitway CC07 project to understand the performance of floating excavation support systems, to validate predictions of numerical analyses and to evaluate new procedures for updating predictions using field measurements obtained during construction. This paper focuses on the numerical predictions and measured performance for the ‘platform section’ of the Courthouse Station, shown in Figure 2. This project is unique among the case studies in Boston, in that detailed Class A predictions were produced during the initial design phase of the temporary works and can be compared with results from more refined analyses performed after completion of the project (i.e., Class C predictions according to the terminology of Lambe, 1973). The results therefore provide a useful basis for evaluating the predictive capabilities and limitations of the numerical analyses.

SITE DESCRIPTION AND INSTRUMENTATION

The MBTA South Boston Piers Transitway Project (Silverline Phase 2) comprises a one-mile long, two-lane subway tunnel with three underground stations connecting South Station to the new Federal Courthouse and World Trade Center in South Boston. The cut-and-cover tunnel and station sections were designed in order to accommodate buses (Silver Line service to Logan Airport) with provisions for future conversion to light rail. The Transitway Project is located within generally level ground that was reclaimed during the period prior to 1916. The ground surface is at elevation El. +34.1m (Metropolitan District Commission Base, MDC datum) with local groundwater table at El. 31.1m (close to mean sea level).

The typical soil profile typically comprises 10-11m of granular/cohesive fill materials and silty sand overlying Boston Blue Clay, Figure 2. The project site investigation report (PBQD, 1998) characterized three surficial units:

1. Granular fill with thickness 1.2m – 2.9m, comprising clean dense sand with $\bar{N}=38 \pm 28$ bpf (n=13).

2. Soft cohesive (hydraulic) fill and/or very soft to medium stiff organic silt with occasional layers of gray silty sand. The combined thickness ranges from 4 to 7.6 m, with $\bar{N}=4 \pm 4$ bpf (n=49).

3. Medium dense, silty sand with thickness ranging from 0.9 to 6.4 m with $\bar{N}=29 \pm 13$ bpf (n=40).
The main unit of Boston Blue Clay ranges in thickness from 22.9 to 37.1m. Physical and index properties of this marine illitic clay have been extensively documented elsewhere. Ladd et al. (1999) report plasticity index, $I_P$ ranging from 20 to 35%, with specific gravity, $G_s = 2.79\pm0.01$, and 50% clay fraction. The liquidity index ($I_L$) generally falls between 0.3 and 0.8, increasing with depth. The pore fluid salt concentration decreases from about 30 g/l (compared to 35 g/l for seawater) at the top of the clay to about 10-15 g/l below El. +6m. Figure 3a summarizes the stress history profile of the clay, based on results of more than 70 1-D consolidation tests (including incremental oedometer, CRS and triaxial feedback-controlled $K_0$-consolidation). The clay can be sub-divided into a lower unit (below El. +6m) that is very lightly overconsolidated with OCR $\approx 1.0 – 1.3$, and an upper clay crust with pre-consolidation rising to $\sigma'_p \approx 700\text{kPa}$ at the top of the unit (OCR $\approx 4.5$).

The BBC is underlain by a glacial till with thickness varying locally from 5m to 7m and soft weathered to hard fractured Cambridge Argillite (see Fig. 8). The glacial till is generally described as a very dense granular material ($\bar{N}=104 \pm 53 \text{ bpf} ; n=44$) containing boulders and cobbles, while core recovery in the argillite ranged from 46 to 100% with the average Rock Quality Designation, $RQD = 20\%$. The groundwater table in the fill fluctuates seasonally in the range El. +29.9 to +31.1m (i.e., slightly below mean sea level). Piezometric measurements in the lower glacial till are 1.8 to 3.0 m below the groundwater table, implying a small head loss (assumed linear) through the BBC layer.

The contract site investigation program included a set of 48 field vane shear tests within the 15 geotechnical borings together with a program of UU triaxial shear tests on tube samples (PB, 1999). These data can be compared with data from a much more extensive research program at the SBST site (Haley & Aldrich, 1993; Ladd et al., 1999). The latter includes tests on samples obtained from fixed piston and block samplers and is the first program to compare properties of $K_0$-consolidated specimens prepared using SHANSEP and Recompression procedures. Results from triaxial and direct simple shear devices also enables direct assessment of the undrained strength anisotropy for BBC. Figure 3b summarizes the undrained shear strength profiles from SHANSEP parameters for undrained triaxial compression ($C_K_U$) and direct simple shear ($C_K_{UDSS}$) modes. For a given mode of shearing, there is a small decrease in $s_u$ through the clay crust to a local minimum at El. +7m ($s_{udss} = 50\text{kPa}$). The undrained shear strength is typically 50-60% higher than that measured in direct simple shear ($s_{udss} = 75\text{kPa}$ at the base of the clay). Results of Recompression tests are consistent with SHANSEP strength profiles (for the triaxial compression mode), but generally show more scatter associated with sample disturbance. The field vane data are in reasonable agreement
with the SHANSEP DSS parameters in the clay crust, but are highly scattered in the lower BBC
(results affected by stress changes in the more sensitive lower BBC) and are much less reliable
for modeling purposes.

There were no laboratory or in situ hydraulic conductivity measurements for BBC in the
site investigations for the CC07 project. The in situ vertical hydraulic conductivity, \(k_v\), at the
was estimated from laboratory CRS tests at the SBST site (Haley & Aldrich, 1993; Whelan,
1995) by extrapolating measured e-logk relations to the in situ void ratio, \(e_\text{v}\). Figure 3c shows
values of \(k_v\) in the range \(3 \times 10^{-8} - 3 \times 10^{-7}\) cm/s, and a trend of decreasing of hydraulic
conductivity with depth.

Figure 4a shows the instrumentation location plan in the central area of the Courthouse
Station, referred to as the ‘Platform section’. There are a series of low-rise buildings located
50-60m from the south wall and one small structure (Seaman’s Chapel) to the north. These
existing buildings are brick construction with heights up to 6 stories and founded on spread
footings. The instrumentation used to monitor the excavation performance for the Platform
section included:

1) Lateral movements of the diaphragm wall by five inclinometers cast within the wall; three in
   the North wall (I-05N1, I-06N1, and I-07N1) and two in the South wall (I-06S1, I-07S1);
2) Lateral deformations in the soil by two inclinometers (I-06N2 and I-06S2) located
   approximately 40 m behind each wall;
3) Surface settlements by surveys of sixteen deflection monitoring points (denoted by DMP)
   and by a series of survey nails located along the tops of the diaphragm wall panels (DMP1);
4) Excavation heave by one multi-point heave gage (HV-6C) located at the center of the
   excavation;
5) Piezometric head measurements from one vibrating wire piezometer inside the excavation
   (PZ-06 C);
6) Strut forces measured by twenty vibrating wire strain gauges distributed at each level of
   bracing and at every second strut line in the Platform section. It should be noted that the
   final design uses single pipe struts for the first three levels of bracing, and pairs of wide
   flange beams at levels 4 and 5 (Fig. 2b).

The aerial photo in Figure 4b shows the temporary works after excavation to final grade
elevation and gives a clear perspective on the adjacent structures and site conditions.
The main objective of the current paper is to evaluate the capabilities of finite element models for predicting the performance of the excavation support system at the Platform section, comparing predictions obtained prior to construction (Class A) versus a posteriori calculations (Class C). In general, constitutive modeling of soil behavior and the selection of model input parameters represent a major source of uncertainty affecting finite element predictions. This project is unusual in this respect, given the proximity of the Courthouse station to the SBST site (Fig. 1), the wealth of in situ and laboratory test data reported in Figure 3, and the extensive prior research in the development and application of advanced effective stress soil models that have been calibrated for Boston Blue Clay. Whittle (1993) and Hashash and Whittle (1996) also reported results of analyses illustrating the factors controlling the performance of floating excavation support systems in BBC.

All of the analyses reported in this paper (both Classes A and C) use the MIT-E3 soil model (Whittle & Kavvadas, 1994; Whittle et al., 1994) to describe the behavior of BBC, while simpler/more conventional elastic, perfectly plastic models are used for the upper soil units, where there is a scarcity of engineering property data. The predictions simulate the coupled flow and deformation in order to represent partial drainage effects within the clay. The Class A predictions are based exclusively on information available at the time of design, they assume a specific time sequence for the excavation process and adopt the conservative assumption that there is no preloading of the cross-lot bracing. In contrast, Class C analyses include refinements in the stratigraphic model (reflecting data from installation of monitoring instruments etc.), include measured pre-loads at each strut level, and simulate the actual timeline of construction events (excavation and support). The following paragraphs detail differences in the Class A and Class C models.

Class A Predictions

Jen (1998) carried out a series of analyses to examine the impact of changes in the diaphragm wall embedment depth, vertical strut spacing, uncertainties in the undrained shear strength and effects of partial drainage on the design of the excavation support system. Figure 2a shows the reference support system for the original design together with the soil profile, referred to as Case A1. The analyses assume plane-strain conditions, with horizontal soil horizons and symmetric loading conditions. The calculations were performed using the ABAQUSTM program with quadrilateral isoparametric finite elements (8-4 node mixed elements for the soils and 8-node solid elements for the elastic diaphragm wall). The diaphragm walls panels are wished-in-place and there is no slip between the concrete panels and adjacent soils (rough interface).
Elastic properties of the struts are given in Table 1. The glacial till serves as a rigid base with constant piezometric head, $H = 30.5\text{m}$. The simulated construction sequence assumes an overdig of $0.6\text{m}$ prior to installation of each level of struts, zero precompression of the struts and a fixed timeframe of 45 days for each excavation level, with an additional 90 days at the final grade elevation (i.e., total construction timeframe is 315 days from the start of excavation for the original design compared to 360 days for the final design).

Table 2 summarizes the key input parameters and models used to describe soil behavior in the Class A predictions. The upper soil units (fill layers and silty sand) are described using a simple linear elastic-perfectly plastic soil model, using correlations with the SPT data (Stroud, 1989) and prior empirical experience from projects in the Boston area (Einstein et al., 1983; Whittle et al., 1993). Boston Blue Clay is modeled using MIT-E3 (Whittle and Kavvadas, 1994) a generalized effective stress soil model that describes rate independent behavior of normally and moderately overconsolidated clays. The model describes a number of important aspects of soil behavior that have been observed in laboratory tests including small strain non-linearity and anisotropic stress-strain-strength properties associated with 1-D consolidation history. Whittle et al. (1994) describe the selection of model input parameters for BBC based on laboratory tests on resedimented clay specimens and show extensive comparisons between computed and measured effective stress paths and stress-strain behavior in a variety of shear modes. They report that the MIT-E3 model gives excellent predictions of the measured undrained stress-strain-strength properties for $K_0$-normally consolidated BBC, but predictions deviate from the measured shear strengths at higher OCR’s. Figure 5 compares the computed undrained shear strength ratios ($\text{s}_d/\sigma'_{vc}$) vs OCR for BBC in a range of shear modes. The results show that MIT-E3 overestimates the undrained strength ratio at $\text{OCR} = 4.0$ by 15-25% compared to measured data from compression (plane strain and triaxial) and simple shear (DSS) modes. In applying the model for analyses of the Transitway excavations, Jen (1998), adjusted the stress history state variables for the model (i.e., OCR, $K_0$) in order to achieve good matching with the undrained shear strength profiles presented in Figure 3b. Appendix A summarizes the input material constants used for the MIT-E3 model (unchanged from Whittle et al., 1994); while Table 2 shows the initial values of the stress history state variables assumed in the current calculations. The analyses effectively assume lower values of $\sigma'_p$ through the clay crust compared to data shown in Figure 3a, in order to achieve more reliable modeling of undrained properties in the clay crust. Table 2 shows that the analyses also assume a constant hydraulic conductivity $k = 4.3\times10^{-8}\text{m/day}$) through the full depth of the BBC unit. This assumption is consistent with measurements by Whittle et al. (2001) at the I-95 test site, but is at the lower end of the range reported by Whelan (1995), Figure 3c.
In actuality, the design of the excavation support was amended prior to construction, as shown in Figure 2b. Corral (2013) carried out Class A analyses for the final design section, using the same modeling assumptions as Jen (1998). These analyses were performed using the Plaxis™ finite element program (Brinkgreve et al., 2008). This program uses 15-3 mixed interpolation triangular elements to model the coupled flow and deformation within the soil mass and 15 node solid elements (i.e., cubic strain interpolation) for the non-porous diaphragm wall. The standard version of the linearly elastic-perfectly plastic soil model in Plaxis uses a conventional Mohr-Coulomb shear strength envelope (referred to as the MC model), with zero dilation, while earlier analyses with ABAQUS use a Drucker-Prager strength criterion. However, the two models are matched to achieve the same shear strength under plane strain conditions. Apart from this difference in detail, the two Class A analyses, referred to as cases A1 and A2, differ only in the specification of the excavation support system (Figs. 2a vs 2b).

Figures 6a and 6b summarize predictions of the lateral wall deformations, bending moments and strut loads for Class A analyses of the original and final designs of the excavation support system (Cases A1 vs A2). The final design has a deeper toe embedment for the diaphragm wall (El. +7.6m vs +10.4m), a thicker wall section (1.22m vs 0.9m) and uses 5 levels of struts (with higher axial stiffness). These modifications are effective in reducing the computing wall deflections by 16-18mm (Fig. 6a) at later stages of the excavation, but have little influence on the mode of deformation of the diaphragm wall. The final design generates larger bending moments associated with higher bending stiffness and greater embedment of the diaphragm wall panels (Fig. 6b). Analysis for the ‘original design’ shows that the allowable load is exceeded at the 4th level of struts (cf. Fig. 2a) and hence, there is clear logic for recommending an extra level of strutting as used in the final design. It can also be seen that the 5th strut level in carries the highest design load (938kN/m, Fig. 6).

Once the excavation reaches the penultimate stage (grade at ~El. +20m), there are similar magnitudes of deflections at the grade elevation and at the toe of the wall. Excavation to the final grade (El. +16.5m) involves large inward movements at the toe. These results highlight the importance of basal stability in the design of the floating support system. Table 3 summarizes the results of stability calculations for the original and final support systems using the average undrained shear strength profile, $s_{u\text{DSS}}$, Figure 3b. The calculations were performed using numerical limit analyses (Ukritichon et al., 2003) and account for the maximum allowable bending capacity of the diaphragm wall panels (Fig. 2). The results show mobilization factors, FS (partial factors of safety on the undrained shear strength) to ensure stability of the support system. Lower and upper bound analyses show that the original design has $FS = 1.21 - 1.37$, while the final design produces a modest increase in the mobilization factor, $FS = 1.26 - 1.42$. 

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The effects of undrained strength anisotropy on stability have been considered based on procedures recommended by Ukritchon et al. (2003). Numerical limit analyses were carried out using the Davis-Christian criterion (Davis & Christian, 1971) to represent variations in the mobilized undrained shear strength with the direction of shearing, based on behavior computed by the MIT-E3 soil model (with OCR profile in Table 2 and a specified shear strain, $\gamma = 1\%$). Table 2 shows that the resulting mobilization factors are very similar to values obtained from isotropic analyses using $s_{udSS}$.

Class C Analyses

Class C analyses (Corral, 2013) incorporate refinements in the model stratigraphy based on inclinometer installation records. Figure 8 shows that the surface of the glacial till dips 5m from the south to north wall, a feature that affects the symmetry assumed in prior analyses. The Class C finite element model is extended into the upper weathered rock to allow for drained relaxation of stresses from the prior assumption of $K_0$-conditions. The model also includes surcharge loads ($q = 25kPa$ acting at the top of the silty sand layer) to represent stresses induced by the 6-storey buildings on the South side of the excavation (Fig. 4). Piezometer monitoring data (from the period prior to construction) show lower piezometric heads in the underlying glacial till ($H = 28.2m$). These are assumed as boundary conditions at the base of the model, with a linear head drop through the clay layer.

The Class C analyses simulate the interpreted time frames of construction (Fig. 8) and average values of pre-loads at each level of strutting (Fig. 2b). However, the model does not simulate details of the chequerboard excavation (and base slab installation), an approximation that affects results computed for the last excavation stage. The Class C analyses assume that the upper soil units (granular and cohesive fills and silty sand) are fully drained layers described. The MC model is used to represent the stress-strain properties of the glacial till and rock layers with input parameters estimated from prior empirical experience (Einstein et al., 1983), Table 2. There are no changes in the MIT-E3 parameters for BBC.

COMPARISON OF MEASUREMENTS AND PREDICTIONS

Figures 9a-f compares the predicted and measured horizontal deflections at the North and South walls of the Platform section at the last three stages of excavation (Stages 4-6). The figures combine data from three inclinometers installed through the North diaphragm wall panels into the underlying argillite (I-05N1, I-06N1 and I-07N1; Fig. 4) with two in the South wall (I-
The figures compare the data with predictions from Class A (case A2) and C analyses.

The inclinometer data show very consistent results along the Platform section. Maximum wall deflections, $\delta_w = 50 – 52$mm (for 4 of the 5 inclinometers) occur at the toe of the diaphragm wall panels at the last stage of excavation. There is little difference in the measured performance of the South and North walls (but slightly more scatter in the data for the latter). The data interpretation assumes that there is zero displacement at the rock-till interface (with less than 6mm measured at the top of the glacial till).

The Class A predictions generally overestimate the inward deformations of the walls above the excavated grade but are in good agreement with the measured toe movements through Stage 5. The analyses overestimate incremental toe movements in the Stage 6 excavations. The Class C analyses are in excellent agreement with the measured wall deflections at all stages of excavation. The differences between the computed Class A and C wall deflections are due, in large part, to modeling of the strut pre-loads. The Class C analysis also represents more accurately the horizontal displacements in the underlying soils due to refinements in modeling the stratigraphy of the basal layers (glacial till and rock), but still tends to overestimate the deformations measured in the lower BBC.

Lateral deformations in the retained soil were only measured intermittently over the course of the project. Figure 10 summarizes the computed horizontal soil deformations at locations approximately 40m from the North and South walls with the measured deformations at the final excavated grade (Stage 6). The data show deformations increasing monotonically from the base of the clay with maximum movements, $\delta_h \approx 26$mm, at the ground surface. The results are in excellent agreement with the Class A predictions, while Class C tend to underestimate movements within the upper soil units. This effect may be attributable to the assumption of drained behavior in the cohesive fill in the Class C model.

Surface settlements were measured by a series of Deflection Monitoring Points (DMP) installed within the pavements and sidewalks surrounding the site (Figs. 4a, b) and at locations along the tops of the diaphragm wall panels. Figures 11a-f compare the computed and measured settlements at each of the last three stages of excavation. There is significant scatter in the measured data among points equidistant from the diaphragm walls, particularly on the south side where DMP’s are located within the sidewalks adjacent to buildings. Settlements are also generally higher on the south side than the north and this may reflect effects such as surcharge loads from the buildings (included in Class C analyses) or differences in the groundwater table across the site (i.e., interference in groundwater seepage conditions due to tunnel construction).
The Class A analyses are in remarkably good agreement with the measured data at final grade elevation. However, these results are closely correlated with inward movements of the diaphragm wall (the top of the diaphragm wall panels are predicted to settle by 10mm; Figs. 11e, f. The Class C analyses estimates smaller maximum surface settlements on the north side than on the south side (15mm vs 20mm; Fig. 11e, f respectively). While the Class C analyses do not match the outer envelope of the measured data, they appear to provide very reasonable predictions of the overall settlement troughs. The Class C analyses predict very small net vertical movements (<3mm) at the tops of the diaphragm walls consistent with the measured data at Stages 5 and 6 (Fig. 11c-f), but also tend to overestimate heave movements at Stage 4 (Figs. 11a, b).

Vertical soil movements beneath the base of the excavation were monitored at 4 elevations within the BBC using a multi-point heave gauge (HV-6C, with a datum at the till-rock interface; Fig. 4a), while pore pressures were obtained nearby (PZ-06C) using vibrating wire piezometers at two elevations. Figure 12 compares the predicted and measured subgrade heave during the last three stages of excavation. The measured data show heave in the upper BBC crust exceeding 70mm at the final stage of excavation. The Class C predictions are in excellent agreement with the measured data at all three stages. The Class A analyses are also in reasonable agreement with the measured data through the fifth stage, but overestimate ground movements in the final stage of excavation. Refinements in the model stratigraphy explain the improved agreement achieved in the Class C predictions.

Figures 13a and 3b compare the computed and measured time histories of pore pressures for piezometers at El. +13.7m and +7.1m, respectively. The measured data show net decrements of pore pressures, \( \Delta u/\sigma_{v0} \approx 0.5 \) (at both elevations) due to the excavation process. The Class A analyses overestimate the initial pore pressures at both depths and clearly provide only a crude estimated timeline of events based on design assumptions. However, pore pressures changes at each phase of excavation are well estimated through stage 5. The analyses overestimate pore pressure changes at stage 6 but reliably show the rebound in pore pressures due to subsequent partial drainage within the clay. Class C predictions match the initial pore pressures and describe accurately the pore pressure time history at both depths. Small discrepancies at stage 6 reflect details of the controlled excavation sequence to final grade that were not present in the finite element model.

Strut loads are interpreted by averaging the measured data from 20 vibrating wire strain gauges installed at different strut lines. All data were properly compensated for temperature effects. The measured data show significantly more variability at strut levels 4 and 5 than in struts 1-3. Class A analyses ignore pre-loading of the struts and hence, generally underestimate
the maximum strut forces (readily apparent at levels 2-4) while overestimating wall deflections. The predicted toe movement during stage 6 excavation generate large strut loads at level 5 and these are in surprisingly good agreement with the measured data. Class C analyses use average measured pre-loads (Fig. 2b) as initial conditions at the time of strut installation. Predictive capabilities can then be assessed by subsequent changes in strut loads and the estimated maximum strut forces. The Class C analyses are in very good agreement with maximum strut loads measured at all levels, but notably overestimate measurements at level 5 (by 20-25%). This result may also be attributed to checkerboard sequencing used during the final stage of excavation, but not considered in the 2D finite element models.

CONCLUSIONS

There is a scarcity of published case studies comparing the measured performance of well-instrumented excavation support systems with numerical predictions. This paper summarizes a detailed set of comparisons that have been achieved for one critical section of the MBTA South Piers Transitway project (platform area of the Courthouse Station), which relied on a floating support system and careful construction and monitoring procedures to mitigate potential problems associated with basal stability safety factor at the lower limit of acceptable limits (during the final stage of excavation).

Research on this project extends back over a period of more than 15 years. The project provided a unique opportunity to evaluate Class A predictions using an advanced effective stress soil model (MIT-E3), with input parameters obtained from a very extensive program of laboratory and in situ tests on the behavior of natural Boston Blue Clay (Ladd et al., 1999). The Class A predictions (after Jen, 1998) involved a series of careful engineering judgments to ensure realistic representation of the undrained shear properties of the 30m deep clay layer. The analyses included careful studies on the effects of partial drainage (over the expected timeframe of construction), strut spacing and toe embedment. Corral (2013) updated the Class A predictions for the as-built structural support system (referred to as the amended design) and carried out detailed Class C predictions for the project. The Class C predictions presented in this paper include refinements in the stratigraphy and in situ pore pressures (based on data from installing monitoring instruments), together with interpreted construction events (time frames of excavation steps and strut pre-loads), but make only nominal changes to the modeling of soil properties. The key findings can be summarized as follows:

1. Class A predictions, using an advanced effective stress soil model, that is well calibrated using high quantity test data, can achieve realistic and consistent predictions of performance
for a braced excavation system including wall deflections, ground movements, and pore pressures. The strut loads can also be well predicted assuming that there is advanced knowledge of pre-load forces at the time of design.

2. Class A predictions performed previously by Jen (1998), which considered several parametric numerical analyses, were extremely useful and relevant for the final geotechnical design. In fact, they enabled construction of the project without resort to expensive ground improvement techniques.

3. The Class C analyses achieve a significant improvement in the prediction of wall deflections and strut loads, and modest improvements in the ground movements (surface settlements and subgrade heave).

4. The Class C analyses described in this paper could have been performed at the time of construction as they involve no iterations on uncertain soil properties. In this respect, they offer conclusive evidence of the predictive accuracy of the proposed finite element models. The results of this study provide strong evidence to support the integrated use of numerical analyses and monitoring data with controlled construction procedures for reducing risks associated with marginal geotechnical stability conditions.

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Figure 1. Site Plan of Courthouse Station, South Piers Transitway project in South Boston (MBTA Contract E02CN14), CA/T tunnels and Special Test Site in South Boston.
Figure 2. Comparison of support systems for Courthouse station excavations, Platform section: a) ‘Original design’ considered in Class A analyses (Jen, 1998); and b) Final (as-built) design
Figure 3. In situ stresses and properties of Boston Blue Clay a) Stress history, b) Undrained shear strength, c) Hydraulic conductivity
Figure 4a. Instrumentation plan for Platform area Courthouse Station
Figure 4b. Aerial view of Courthouse Station excavations, September 2001
Figure 5. Comparison of MIT-E3 predicted and measured undrained shear strength ratios for resedimented Boston Blue Clay (after Whittle et al., 1994)
Figure 6. Summary of Class A predictions (no preloading of struts)
a) Failure mechanism, ‘Original design’

b) Failure mechanism, final design

Figure 7. Upper bound stability analyses for original and final designs from numerical limit analyses
Figure 8. Cross-section for Class C analyses
Figure 9. Comparison of computed and measured lateral wall deflections at last three stages of excavation.
Figure 10. Comparison of computed and measured lateral soil deformations at locations 40m from the diaphragm walls.
Figure 11. Comparison of predicted and measured surface settlements at excavation Stages 4-6
Figure 12. Comparison of predicted and measured below-grade heave at center of excavation for stages 4-6.
Figure 13. Comparison of predicted and measured pore water pressure-time response in clay beneath center of excavation
Figure 14. Comparison of computed and measured maximum strut loads at each level of strutting.
Table 1. Stiffness properties of struts

<table>
<thead>
<tr>
<th>Case</th>
<th>Strut Level</th>
<th>Section</th>
<th>Ave. Spacing, s (m)</th>
<th>Cross-Sect. Area, A (cm²)</th>
<th>Axial Stiffness, EA/s (MN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>1-4</td>
<td>φ=0.76m w=1.9cm</td>
<td>6.7</td>
<td>445</td>
<td>1330</td>
</tr>
<tr>
<td>A2 &amp; C</td>
<td>1</td>
<td>φ=0.61m w=1.6cm</td>
<td></td>
<td>296</td>
<td>920</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>φ=0.91m w=1.6cm</td>
<td></td>
<td>448</td>
<td>1400</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>φ=0.91m w=2.5cm</td>
<td></td>
<td>709</td>
<td>2200</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>2-W36x135</td>
<td>6.45</td>
<td>512</td>
<td>1600</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>2-W36x230</td>
<td></td>
<td>872</td>
<td>2700</td>
</tr>
</tbody>
</table>

Note: Young’s modulus of steel, E = 200GPa
### Table 2. Soil models and input parameters for Class A analyses

<table>
<thead>
<tr>
<th>Unit</th>
<th>Top El. (m)</th>
<th>Model</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$c'$ (kPa)</th>
<th>$\phi_{ps}$ ($^\circ$)</th>
<th>OCR</th>
<th>$K_0$</th>
<th>$G/\sigma'_{v0}$</th>
<th>$v'$</th>
<th>$k$ (m/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Granular Fill</td>
<td>34.1</td>
<td>EPP</td>
<td>18.9</td>
<td>0</td>
<td>30</td>
<td>--</td>
<td>0.50</td>
<td>35</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Cohesive Fill</td>
<td>32.3</td>
<td>EPP</td>
<td>18.1</td>
<td>[0.22]</td>
<td>--</td>
<td>--</td>
<td>0.75</td>
<td>20</td>
<td>0.3</td>
<td>0.009</td>
</tr>
<tr>
<td>Silty Sand</td>
<td>25.6</td>
<td>EPP</td>
<td>18.9</td>
<td>0</td>
<td>35</td>
<td>--</td>
<td>0.50</td>
<td>35</td>
<td>0.3</td>
<td>0.3</td>
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<tr>
<td>BBC</td>
<td>21.9</td>
<td>MIT-E3</td>
<td>18.1</td>
<td>Appendix A</td>
<td>3.00</td>
<td>0.84</td>
<td>Appendix A</td>
<td>4.3x10$^{-5}$</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>19.5</td>
<td></td>
<td></td>
<td></td>
<td>2.52</td>
<td>0.77</td>
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<td></td>
<td>17.1</td>
<td></td>
<td></td>
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<td>2.14</td>
<td>0.71</td>
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<td></td>
<td>14.6</td>
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<td>1.79</td>
<td>0.66</td>
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<td></td>
<td>12.2</td>
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<td>1.46</td>
<td>0.61</td>
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<td></td>
<td>9.8</td>
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<td></td>
<td></td>
<td>1.25</td>
<td>0.57</td>
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<tr>
<td></td>
<td>7.3</td>
<td></td>
<td></td>
<td></td>
<td>1.0</td>
<td>0.53</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Glacial Till</td>
<td>-7.7</td>
<td>EPP</td>
<td>18.9</td>
<td>0</td>
<td>43</td>
<td>--</td>
<td>1.0</td>
<td>250</td>
<td>0.3</td>
<td>0.09</td>
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<tr>
<td>Argillite</td>
<td>Elastic</td>
<td>21.8</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>1.0</td>
<td>10000</td>
<td>0.3</td>
<td>0.009</td>
</tr>
</tbody>
</table>

**Notes:**
1. EPP – Elastic Perfectly Plastic model (Drucker-Prager yield in ABAQUS; Mohr-Coulomb Yield in Plaxis)
2. Shaded cells – alternate properties used in Class C analyses
3. Concrete diaphragm walls: $E = 2.26 \times 10^4$ MPa, $v' = 0.15$
Table 3. Mobilization factors on undrained shear strength of soil for stability of support systems

<table>
<thead>
<tr>
<th>Design</th>
<th>Original</th>
<th>Final/Amended</th>
<th>Final/Amended</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay shear strength</td>
<td>Isotropic - $s_{udSS}$</td>
<td>Isotropic- $s_{udSS}$</td>
<td>Mobilized anisotropic*</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>At $\gamma = 1.0%$</td>
</tr>
<tr>
<td>Lower Bound</td>
<td>1.211</td>
<td>1.259</td>
<td>1.241</td>
</tr>
<tr>
<td>Upper Bound</td>
<td>1.368</td>
<td>1.415</td>
<td>1.395</td>
</tr>
</tbody>
</table>

* Based on MIT-E3 predictions using OCR profile in Table 2
<table>
<thead>
<tr>
<th>Test Type</th>
<th>Parameter / Symbol</th>
<th>Physical contribution/meaning</th>
<th>Boston Blue Clay (BBC)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-D Consolidation</td>
<td>e₀</td>
<td>Void ratio at reference stress on virgin consolidation line</td>
<td>0.988</td>
</tr>
<tr>
<td>(Oedometer, CRS, etc.)</td>
<td>λ</td>
<td>Compressibility of virgin normally consolidated clay</td>
<td>0.184</td>
</tr>
<tr>
<td></td>
<td>c</td>
<td>Non-linear volumetric swelling behavior</td>
<td>22.0</td>
</tr>
<tr>
<td></td>
<td>n</td>
<td>Irrecoverable plastic strain</td>
<td>1.6</td>
</tr>
<tr>
<td>K₀-Oedometer or K₀-Triaxial</td>
<td>K₀NC</td>
<td>K₀ for virgin normally consolidated clay</td>
<td>0.53</td>
</tr>
<tr>
<td></td>
<td>2G/K</td>
<td>Ratio of elastic shear to bulk modulus (Poisson’s ratio for initial unload)</td>
<td>1.05</td>
</tr>
<tr>
<td>Undrained Triaxial</td>
<td>φ’&lt;sub&gt;TC&lt;/sub&gt;</td>
<td>Critical state friction angles in triaxial compression and extension (large strain failure criterion)</td>
<td>33.4°</td>
</tr>
<tr>
<td></td>
<td>φ’&lt;sub&gt;TE&lt;/sub&gt;</td>
<td></td>
<td>45.9°</td>
</tr>
<tr>
<td>OCR=1; CKoUC</td>
<td>c</td>
<td>Undrained shear strength (geometry of bounding surface)</td>
<td>0.866</td>
</tr>
<tr>
<td>OCR=1; CKoUE</td>
<td>Sᵢ</td>
<td>Amount of post-peak strain softening in undrained triaxial compression</td>
<td>4.5</td>
</tr>
<tr>
<td>OCR=2, CKoUC</td>
<td>ω</td>
<td>Non-linearity at small strains in undrained shear</td>
<td>0.07</td>
</tr>
<tr>
<td></td>
<td>γ</td>
<td>Shear induced pore pressure for OC clay</td>
<td>0.5</td>
</tr>
<tr>
<td>Parameter</td>
<td>Value</td>
<td></td>
<td></td>
</tr>
<tr>
<td>---------------------------------</td>
<td>-------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear wave velocity $\kappa_0$</td>
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<tr>
<td>Small strain compressibility at load reversal</td>
<td>0.001</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Drained Triaxial $\psi_0$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rate of evolution of anisotropy (rotation of bounding surface)</td>
<td>100</td>
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</tr>
</tbody>
</table>

Note:

1. Input parameters based on laboratory tests on resedimented BBC.