ReAnalysis of Deep Excavation Collapse Using a Generalized Effective Stress Soil Model

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Re-analysis of Deep Excavation Collapse Using a Generalized Effective Stress Soil Model

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ABSTRACT

This paper re-analyzes the well-documented failure of a 30m deep braced excavation in underconsolidated marine clay using an advanced effective stress soil model (MIT-E3). The collapse of the Nicoll Highway during construction of cut-and-cover tunnels for the new Circle Line in Singapore has been extensively investigated and documented. All prior analyses of the collapse have relied on simplified soil models with undrained strength parameters based on empirical correlations and piezocone penetration data. The current analysis use results from high quality consolidation and undrained triaxial shear tests that were only available after completion of the public inquiry. The current analyses achieve very reasonable estimates of measured wall deflections and strut loads using model parameters derived directly from the laboratory tests. The analyses confirm prior interpretations of the failure mechanism but provide a more rational basis for the modeling of soil-structure interaction.

INTRODUCTION

The collapse of the Nicoll Highway during excavations for the cut-and-cover tunnels for the new Circle Line in Singapore (Phase 1 contract C824) has been extensively documented in a Committee of Inquiry report (COI, 2005). Many local and international experts contributed to this report and have subsequently published detailed interpretations of the failure (e.g., Yong et al., 2006; Endicott, 2006; Davies et al., 2006). One key aspect was the under-design of the temporary lateral earth support system. Figure 1a shows the design for the (intended) 33.3m deep excavation comprising 0.8m thick diaphragm wall panels that extend through deep layers of Estuarine and Marine clays (Kallang formation) and are embedded a minimum of 3m within the underlying Old Alluvium (layer SW-2). The walls were to be supported by a total of ten levels of pre-loaded, cross-lot bracing and by two relatively thin rafts of continuous Jet Grout Piles (JGP). The Upper JGP raft was a sacrificial layer that was excavated after installation of the 9th level of struts. Collapse occurred on April 20th 2004 following excavation of the Upper JGP (to an elevation of approximately 72.3m RL, Fig. 1).
The design of the temporary lateral earth support system was based on a table of geotechnical design parameters (GIM, August 2001). This table included the unit weights, $K_0$ coefficients, hydraulic conductivities, $k$, elastic moduli, $E$, and both the Mohr-Coulomb (drained) effective stress strength parameters ($c'$, $\phi'$) and undrained shear strength profiles, $s_u(z)$ for all of the main soil units and JGP layers. Many of these parameters were based on prior local experience (e.g., Bo et al., 2003; Tan et al., 2003; Chiam et al., 2003; Li & Wong, 2001).

Figure 1(a). Cross-section of excavation support system design section, and (b) undrained shear strength profiles

Piezocone penetration data were the only reliable site-specific information on undrained shear strengths available at the time of design. Figure 1b compares the undrained shear strength profile specified in the GIM table with results from 4 piezocone tests interpreted using a cone factor $N_{KT} = 14$. The results show good agreement between the GIM and piezocone strengths in the Upper unit of the Marine Clay (UMC). However, the piezocone results also suggest that the Lower Marine Clay (below 75mRL) is weaker than the design strength profile. Whittle and Davies (2006) have attributed this to underconsolidation of the Lower Marine Clay associated with 5m of fill used to reclaim the land in the 1970’s. This explanation assumes that the underlying units of Old Alluvium have low bulk permeability and/or low recharge potential.

The design of the lateral earth support system was based on finite element analyses of soil-structure interaction using an elastic-perfectly plastic (Mohr-
Coulomb, MC) model for the soil behavior. The analyses simulated undrained shear behavior of the clay layers using drained effective stress strength parameters (c’, φ’). This approach, referred to as Method A (COI, 2005), led to gross overestimation of the undrained shear strength in the analyses (Fig. 1b). As a result the designers underestimated the wall deflections and bending moments and under-designed the bending capacity of the diaphragm wall and thickness of the two JGP layers.

Ironically, most of the experts involved in the investigations of the collapse used the same finite element program and MC constitutive model to diagnose the failure mechanism. These experts used total stress strength parameters (su = c’, φ’ = 0°) to represent directly the expected undrained strength profiles (e.g., GIM, AJW&RVD; Fig. 1b). These Method B analyses were able to describe, to a reasonable first order approximation, the measured lateral wall deflections and strut loads. They also provided the basis for explaining the collapse mechanism in which the brittle failure of the 9th level strut-waler connections led to a redistribution of lateral earth pressures that could not be supported by the bracing system and led to catastrophic failure.

Extensive post-failure site investigation programs were carried out to resolve uncertainties associated with the complex stratigraphy (which includes a relic deep relic channel through the Old Alluvium). A detailed program of high quality laboratory consolidation and shear strength testing on high quality samples of marine clay was also performed (Kiso-Jiban, 2004). None of these data were analyzed in detail at the time of the inquiry but were included in a revised design manual (Amberg, 2005). This paper presents a re-analysis of the excavation performance based on the post-failure laboratory test program. The behavior of the Upper (UMC) and Lower (LMC) Marine Clay units is represented by the MIT-E3 model (Whittle and Kavvadas, 1994) which is able to simulate the anisotropic effective stress-strain-strength properties measured in the tests.

MODEL CALIBRATION

The MIT-E3 soil model (Whittle & Kavvadas, 1994) was developed to simulate the effective stress-strain-strength behavior of normally and moderately overconsolidated clays. The model describes a number of important aspects of soil behavior which have been observed in laboratory tests on K0-consolidated clays including: 1) small strain non-linearity following a reversal of load direction; 2) hysteretic behavior during unload-reload cycles of loading; 3) anisotropic stress-strain-strength properties associated with 1-D consolidation history and subsequent straining; 4) post-peak, strain softening in undrained shear tests in certain modes of shearing on normally and lightly overconsolidated clays; and 5) occurrence of irrecoverable plastic strains during cyclic loading and shearing of overconsolidated clays. The model also has a number of key restrictions: It uses a rate independent formulation and hence, does not describe creep, relaxation or other strain rate dependent properties; and 2) it assumes normalized soil properties (e.g., the strength and stiffness are proportional to the confining pressure at a given overconsolidation ratio, OCR) and hence, does not describe complex aspects of soil behavior associated with cementation.
Calibration of the model for UMC and LMC clays follows the general procedure proposed by Whittle et al. (1994). Table 1 summarizes these input parameters, their physical meanings within the model formulation and laboratory tests from which they can be obtained, together with parameters selected for UMC and LMC units. The parameters have been derived principally from a set of 1-D consolidation tests (Fig. 2) and $K_0$-consolidated undrained triaxial shear tests (Fig. 3) on specimens reconsolidated to the in situ stress conditions.

Table 1: Input Parameters for MIT-E3 Constitutive Soil Model: UMC & LMC

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Parameter/ Symbol</th>
<th>Physical contribution/meaning</th>
<th>Upper Marine Clay (UMC)</th>
<th>Lower Marine Clay (LMC)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-D Consolidation (Oedometer, CRS, etc.)</td>
<td>$e_0$</td>
<td>Void ratio at reference stress on virgin consolidation line</td>
<td>1.80</td>
<td>1.60</td>
</tr>
<tr>
<td></td>
<td>$\lambda$</td>
<td>Compressibility of virgin normally consolidated clay</td>
<td>0.380</td>
<td>0.370</td>
</tr>
<tr>
<td></td>
<td>$C$</td>
<td>Non-linear volumetric swelling behavior</td>
<td>10.0</td>
<td>9.0</td>
</tr>
<tr>
<td></td>
<td>$p$</td>
<td></td>
<td>1.50</td>
<td>1.50</td>
</tr>
<tr>
<td></td>
<td>$h$</td>
<td>Irrecoverable plastic strain</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>$K_0$-Oedometer or $K_0$-Tria xial</td>
<td>$K_{0NC}$</td>
<td>$K_0$ for virgin normally consolidated clay</td>
<td>0.52</td>
<td>0.52</td>
</tr>
<tr>
<td></td>
<td>$2G/K$</td>
<td>Ratio of elastic shear to bulk modulus (Poisson’s ratio for initial unloading)</td>
<td>0.94</td>
<td>0.94</td>
</tr>
<tr>
<td>Undrained Triaxial Shear Tests: OCR=1; CKoUC</td>
<td>$\phi^{TC}$</td>
<td>Critical state friction angles in triaxial compression and extension (large strain failure criterion)</td>
<td>32.4°</td>
<td>27.0°</td>
</tr>
<tr>
<td></td>
<td>$\phi^{TE}$</td>
<td></td>
<td>33.8°</td>
<td>27.1°</td>
</tr>
<tr>
<td>OCR=1; CKoUE</td>
<td>$c$</td>
<td>Undrained shear strength (geometry of bounding surface)</td>
<td>0.96</td>
<td>0.96</td>
</tr>
<tr>
<td>OCR=2; CKoUC</td>
<td>$S_n$</td>
<td>Amount of post-peak strain softening undrained triaxial compression</td>
<td>5.0</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>$\varepsilon_0$</td>
<td>Non-linearity at small strains in undrained shear</td>
<td>0.40</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td>$\gamma$</td>
<td>Shear induced pore pressure for OC clay</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Shear wave velocity</td>
<td>$\kappa_0$</td>
<td>Small strain compressibility at load reversal</td>
<td>0.0094</td>
<td>0.0094</td>
</tr>
<tr>
<td>Drained Triaxial</td>
<td>$\psi_0$</td>
<td>Rate of evolution of anisotropy (rotation of bounding surface)</td>
<td>100.0</td>
<td>100.0</td>
</tr>
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</table>

The compressibilities of the normally consolidated UMC and LMC units are well-characterized virgin consolidation lines with $\lambda = 0.37 \text{ – } 0.38$, Figure 2a. The upper marine clay generally has higher in situ void ratio ($e = 1.7 \text{ – } 1.9$) than the lower unit ($e = 1.5 \text{ – } 1.6$). The marine clays show significant elastic rebound when unloaded. Figure 2b shows that recoverable axial strains, $\Delta e_a = 10\text{-}12\%$ when the effective stress is reduced by one order of magnitude ($\xi_v = OCR = 10$). This behavior is consistent with laboratory measurements of the maximum shear modulus, $G_{max}$ (from bender elements), reported by Tan et al. (2003). The Authors have used these data to estimate the model input parameter, $\kappa_0$, and then selected input values of $C$, $n$ (Table 1) to the swelling data as shown in Figure 2b.
A series of CAU normally consolidated triaxial compression and extension tests were performed on specimens from 4 depths within the UMC and LMC units, Figure 3.

**Figure 2:** Compression and swelling properties of the Upper and Lower Marine Clays

**Figure 4:** Comparison of measured undrained shear behavior from laboratory CAU compression and extension tests on normally consolidated UMC and LMC specimens with numerical simulations using the MIT-E3 model
All of the specimens were consolidated to a common lateral stress ratio, $K_0 = 0.50$ prior to shearing. The measured data show a significant difference in the average undrained triaxial compression strength ratios measured in these tests, $s_{uTC}/\sigma'_vc = 0.30$ vs $0.27$ for the UMC and LMC units, respectively. The data also show that UMC specimens mobilize higher friction angles when sheared to large strains (in both compression and extension), $\phi' = 32.4^\circ -33.8^\circ$ vs $27.0^\circ - 27.1^\circ$ for UMC and LMC. The UMC exhibits higher undrained strength anisotropy, $s_{uTE}/s_{uTC} = 0.60 – 0.66$ compared to LMC (0.80 – 0.88) and both exhibit relatively modest post-peak softening in compression shear modes for $\varepsilon_a > 2\%$.

Details of the measured effective stress paths and shear stress-strain properties are well characterized by MIT-E3 through model input parameters $c$, $S_t$, $\phi'_{TC}$, $\phi'_{TE}$, $\omega$ and $\gamma$ (Table 1). The remaining parameters in Table 1 have been estimated from prior studies on similar clays.

**FINITE ELEMENT MODEL**

The numerical simulations of excavation performance have been carried out focusing on one specific cross-section (within the collapse zone) corresponding to the location of the instrumented strut line S335, Figure 4. Loads in each of the nine levels of struts installed at S335 were measured through sets of three strain gauges. These data have been extensively validated by each of the expert witnesses for the public inquiry (e.g., Davies et al., 2006). Measurements of the lateral wall movements at this section are obtained from inclinometer I-65 (installed through the north diaphragm wall panel) and I-104 located in the soil mass 1.5 – 2.0m outside the South wall.

![Figure 4: Plan showing the structural support system and 9th level strutting and monitoring instrumentation](image)
complex that the design section indicated in Figure 1. The base of the LMC dips notably to the south. This is part of a relic channel in the underlying Old Alluvium that was highlighted by Whittle and Davies (2006). On the south side, the LMC directly overlies the Old Alluvium, while units of fluvial sand, F1, and estuarine clay (E) separate the Marine Clay and OA on the north side. The post-failure investigations have established that the OA has relatively low bulk hydraulic conductivity, while the F1 layer has relatively limited extent and no ready source of recharge (although there is a hydraulic connection across the wall due to the absence of a diaphragm wall panel between S336 – S337 in Fig. 4). These details were critical in establishing that failure of the excavation was not caused by hydraulic uplift. The lateral earth support design includes two layers of continuous jet grout pile (JGP) rafts that were intended to provide additional passive resistance below the formation. At section S335 it is unlikely that the lower JGP raft is continuous within the Old Alluvium, as installation jetting parameters for the jet grout columns were based on parameters calibrated to marine clay conditions. Hence, the section shows a truncation of the lower JGP raft at the North wall.

![Figure 5: S335 Section geometry used in FE model](image)

Section S335 has been modeled using the Plaxis™ program. The MIT-E3 model has recently been integrated within the kernel of Plaxis (Akl, Bonnier; pers. comm., 2008). Following Whittle and Davies (2006), the current numerical simulations assume that the groundwater table in the Fill is at 100.5m RL and that there is small excess pressure in the underlying LMC and OA units (piezometric head, H = 103m). The UMC and LMC units\(^1\) are modeled using the MIT-E3 model with parameters listed in Table 1, while engineering properties of all other soils and

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\(^1\) The lower Estuarine clay (E, Fig. 5) is assumed to have the same properties and behavior as the LMC
JGP rafts are simulated using the Mohr-Coulomb (MC) model with parameters reported in the prior studies (COI, 2005). In order to apply the MIT-E3 effective stress soil model it is essential to specify carefully the in situ effective stress profile and the initial OCR. The current analyses assume \( \sigma'_p/\sigma'_v_0 = 1.0 \) in both UMC and LMC units (Fig. 6a). When combined with the assumed pore pressure conditions, this implies that the marine clays are slightly under-consolidated. The in-situ stresses also deviate from \( K_0 \)-conditions due to the inclined stratigraphy. This is modeled using a standard drained relaxation of stress procedure within Plaxis.

Figure 6b summarizes the anisotropic undrained shear strength profiles within the marine clays obtained using the MIT-E3 model for three standard modes of plane strain shearing. The undrained plane strain active and passive strengths bound the best estimate profile recommended by Whittle and Davies (2006), based on their interpretation of piezocone tests (this assumes \( s_{udSS}/\sigma'_v_0 = 0.21 \) for normally consolidated Singapore marine clay, after Tan et al., 2003). It is interesting to note that the undrained shear strength predicted by MIT-E3 in the DSS mode is 5-7kPa lower than the best estimate used in the prior MC analyses within the LMC.

![Figure 6: Comparison of in situ stresses and undrained strengths of marine clay used in FE model](image-url)

a) In situ stresses  
b) Undrained strengths in marine clay

All other parameters for the lateral earth support system including the as-built diaphragm wall embedment, capacity of the critical strut-water connections and pre-load of the struts are based on prior interpretation of the construction records (Bell &
Figure 7: Comparison of computed and measured lateral wall deflections at Section S335, March – April 2004
RESULTS

Figure 7 compares predictions of lateral wall deflections from the current analyses with measured data from the two inclinometers (I-104, I-65) and with results of prior analyses (marked as MC) performed by Whittle & Davies (2006). The results are shown at three times during the month preceding the collapse (with excavation depths 24.6m, 27.6 and 30.6m). The current analyses predict very well the maximum lateral wall deflection on the south side of the excavation including the large deflections associated with removal of the upper JGP layer (April 17-20). At this stage, a plastic hinge formed in the South wall (at a depth of 32m) and there is very large rotation of the toe. The current analyses also describe very well the maximum lateral wall deflection on the north side through March. The analyses tend to overestimate inward movements of both walls within the upper 10-15m of the bracing system. This may be attributed to the assumption that the UMC is normally consolidated, while the pre-consolidation data show a small OCR in this layer (Fig. 6a). The analysis predicts significant lateral displacements at the toe of the north wall in April 2004 (70mm at time of failure on April 17-20). In contrast, inclinometer I-65 suggests that the north diaphragm wall panel remains well anchored. The net effect is that the analysis underestimates the deflections and flexure in the lower part of the north wall during April. This result is largely related to the complex stratigraphy and assumed truncation of the lower JGP at the north wall.

The current analyses using MIT-E3 predict larger inward wall deflections than the prior MC analyses and are in rather better agreement with the measured data. This result is encouraging as the current analyses are based on calibration of a complex constitutive model using laboratory test data (rather than a best estimate of a design strength line). However, it is clear that certain features of the measured data such as the toe fixity on the north wall are difficult to interpret and are not controlled by the properties of the marine clay. Similarly, the current analyses do require additional judgment in the selection of the OCR profile.

It is generally agreed that collapse of the Nicoll Highway initiated when the 9th level strutting failed due to sway buckling of the strut-waler connections. Overloading of the strut-waler connections occurred due to the absence of splays that had been designed for all struts (see Fig. 4). The strut-waler connections exhibited a brittle post-peak load response due to a mechanism of ‘sway buckling’ that was associated with the use of c-channel stiffeners at the strut-waler connections in levels 7-9 of the bracing system (this was a revised design used during construction; Bell & Chiew, 2006). Collapse occurred as the bracing system was unable to handle loads transferred upward through the bracing system. Figure 8 summarizes predictions of the strut loads at levels 7-9 on prior to collapse (April 20, 2004). The results show very reasonable agreement between the computed and measured loads in strut levels 7 and 8. The current analyses are also in close agreement with loads obtained by Whittle and Davies (2006) using the MC model. Both sets of analyses predict that the capacity of the 9th level strut-waler connection is fully mobilized at this stage of excavation (30.6m deep) immediately following removal of the upper JGP raft. In contrast the measured strut loads are much smaller. This is an inconsistency noted by all the experts to the public inquiry (COI, 2005). Hence, it can be concluded that the
current analyses with MIT-E3 are able to predict the onset of collapse consistent with prior MC analyses but do not shed any insight to explain the measured loads at level 9.

Figure 8: Comparison of computed and measured strut loads for excavation to 30.6m (April 17-20, 2004)

CONCLUSIONS

The Authors have re-analyzed the performance of the lateral earth support system for a critical instrumented section, S335, of the cut-and-cover excavations at the site where the Nicoll Highway collapsed in 2004. Engineering properties of the key Upper and Lower marine clay units have been modeled using the generalized effective stress soil model, MIT-E3, with input parameters calibrated using laboratory test data obtained as part of the post-failure site investigation. The model predictions are evaluated through comparisons with monitoring data and through comparisons with results of prior analyses using the Mohr-Coulomb (MC) model (Whittle & Davies, 2006). The MIT-E3 analyses provide a modest improvement in predictions of the measured wall deflections compared to prior MC calculations and give a consistent explanation of the bending failure in the south diaphragm wall and the overloading of the strut-waler connection at the 9th level of strutting. The current analyses do not resolve uncertainties associated with performance of the JGP rafts, movements at the toe of the north-side diaphragm wall or discrepancies with the measured strut loads at level 9. However, they represent a significant advance in predicting excavation performance based directly on results of laboratory tests.
compared to prior analyses that used generic (i.e., non site-specific) design isotropic strength profiles.

REFERENCES


