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

by

Gonzalo Andrés Corral Jofré
B.Sc. in Civil Engineering (2000)
Structural Engineer (2001)
Pontificia Universidad Católica de Chile

M.Sc. in Geotechnical Engineering (2007)
Universidad de Chile

Submitted to the Department of Civil & Environmental Engineering
In Partial Fulfillment of the Requirements for the degree of

CIVIL ENGINEER

at the
Massachusetts Institute of Technology
June 2010

© 2010 Massachusetts Institute of Technology
All rights reserved

Signature of Author......................................

Department of Civil & Environmental Engineering

Certified by...................................................

Andrew J. Whittle
Professor of Civil and Environmental Engineering
Thesis Supervisor

Accepted by..............................................

Daniele Veneziano
Chairman, Department Committee on Graduate Students
Re-analysis of Deep Excavation Collapse
Using a Generalized Effective Stress Soil Model

by

Gonzalo Andrés Corral Jofré

Submitted to the Department of Civil and Environmental Engineering on May 21, 2010,
in Partial Fulfillment of the Requirements for the Degree of Civil Engineer

Abstract

This thesis re-analyzes the well-documented failure of a 30m deep braced excavation
underconsolidated marine clay. Prior analyses of the collapse of the Nicoll Highway have relied
on simplified soil models with undrained strength parameters based on empirical correlations and
piezocone penetration data. In contrast, the current research simulates the engineering properties
of the key Upper and Lower Marine Clay units using a 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.

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.

Thesis Supervisor: Prof. Andrew J. Whittle
Title: Professor of Civil and Environmental Engineering
Acknowledgements

First and foremost, I would like to thank my supervisor, Professor Andrew J. Whittle, for giving me the opportunity of working with him on this fascinating project. Thanks for the guidance and advices during this research work. This has undoubtedly been a remarkable learning process.

I would also like to thank my friend and colleague, Sherif Akl for playing an important role on a part of this thesis, and also to my friend Dr. Maria Nikolinakou.

Additionally, I would like to express my sincere gratitude to Dr. Lucy Jen, and Dr. John T. Germaine for being always willing to help.

Finally to my family: Elisa, Eduardo & Ivette, Loreto, Cristóbal, Josefa, Tomás, Dominga, Antonia, Francisca, and Amalia for being a continuous support and appreciation of each personal goal and interest I have had. Specially, I would like to thank, Natalia, for being a tremendous support and encouragement during this MIT experience.
# TABLE OF CONTENTS

Abstract ................................................................................................................................. 3

Acknowledgements .............................................................................................................. 5

Table of Contents ................................................................................................................ 7

List of Figures ...................................................................................................................... 10

List of Tables ....................................................................................................................... 15

1 Introduction....................................................................................................................... 17

2 Project & Collapse Review ............................................................................................ 20

2.1. Overview..................................................................................................................... 20

2.2. Contract C-824 ......................................................................................................... 20

2.3. Events Leading Up the Collapse .............................................................................. 22

2.4. Collapse Causes Summary ....................................................................................... 23

3 Site Conditions ................................................................................................................ 41
6.2. Comparison of Computed and Measured Wall Deflection ........................................ 110
6.3. Computed Bending Moments in Diaphragm Wall .................................................. 112
6.4. Computed and Measured Strut Loads ..................................................................... 112
6.5. Vertical Settlements ............................................................................................... 113

7 Conclusions & Recommendations .............................................................................. 134

7.1. Summary .................................................................................................................. 134
7.2. Conclusions ............................................................................................................. 134
7.3. Recommendations ................................................................................................. 135

List of References ......................................................................................................... 136
LIST OF FIGURES

Figure 1-1: Nicoll Highway Collapse – April 20th 2004 (COI, 2005) .............................................. 19
Figure 2-1: Subway system in Singapore, showing new Circle Line project in 2006 ............. 27
Figure 2-2: Overview of Circle Line Stages (COI, 2005) ......................................................... 28
Figure 2-3: Overview of Circle Line Stage 1: CCL1 (COI, 2005) ............................................. 29
Figure 2-4: Chart showing the Relationship of Parties Involved in C824 (COI, 2005) .......... 30
Figure 2-5: Contract C824 involved approximately 2.8 km of route and included the construction of Nicoll Highway & Boulevard Stations, and the linking tunnels (Davies et al., 2006) ................................................................. 31
Figure 2-6: Construction Principle of the Jet Group Pile – JGP – Slab (COI, 2005) ............. 33
Figure 2-7: M3 typical design cross-section of excavation support system (Whittle and Davies, 2006) ......................................................................................................................... 34
Figure 2-8: Plan showing the structural support system and 9th level strutting and monitoring instrumentation (Corral & Whittle, 2010) ................................................................. 35
Figure 2-9: Diagram: Collapse Causes interpreted by ARUP (ARUP, 2005) ......................... 36
Figure 2-10: Mohr-Coulomb Failure Model: Undrained Shear Strengths derived from Methods A, B, C & D used in the C824 Project (COI, 2005) ........................................... 38
Figure 2-11: Lateral Wall Defection comparison between Method A & B (COI, 2005) ...... 40
Figure 3-1: Geology of Singapore showing the North East Line Location (Shirlaw et al., 2000) ................................................................................................................................. 48
Figure 3-2: Results of recent studies of paleo-channels beneath Kallang formation (a) Transition from Bedrock Controlled Channels into Meandering Channels (b) Location of Confirmed Palaeochannels, (c) Palaeochannels Likelihood Mapping (Mote et al. 2009) ................................................................. 49
Figure 3-3: Typical Profile in Kallang Formation Area (Davies, 1984) ........................................ 50

Figure 3-4: Land Reclamations in the 30's & 70's (Davies at al., 2006) ........................................ 51

Figure 3-5: Prior Reclamation of Land: Aerial Photo taken 5th April 1969 (Davies at al., 2006) ................................................................................................................................. 52

Figure 3-6: Location of Boreholes, Piezocones & Diaphragm Wall Panels in the M3 section. Blue and red circles and triangles represent the boreholes and piezocones for pre-tender and post-tender, accordingly. (Whittle & Davies, 2006) .................. 53

Figure 3-7: Cross Section A-A, showing the Required Final Depth of the Excavation and Variations of OA & LMC (Davies at al. 2006) ................................................................. 54

Figure 3-8: Cross Section B-B, showing the Required Final Depth of the Excavation and Variations of OA & LMC (Davies at al. 2006) ................................................................. 55

Figure 3-9: Contours of base of Lower Marine Clay, in m, RL (Whittle & Davies, 2006) .... 56

Figure 3-10: Contours of top of Old Alluvium (for \( N_{sp}>30 \)), in m, RL (Whittle & Davies (2006))................................................................................................................................. 57

Figure 3-11: Locations of Piezometer & Settlement Points and Reclamation History (COI, 2005) ................................................................. 58

Figure 3-12: Ground Surface Settlement Measured prior to Construction (Whittle & Davies, 2006) .................................................................................................................. 59

Figure 3-13: Undrained shear strength profiles (Corral & Whittle, 2010) ......................... 60

Figure 3-14: Construction Sequence for JGP slabs (COI, 2005)........................................ 61

Figure 3-15: Summary of As-Built South & North Diaphragm Wall Panel Embedment in M3 area (Whittle & Davies, 2006) ........................................................................... 62

Figure 3-16: Lateral Wall Deflection Measurements for Excavation Levels 6, 7, 8, & 9 (Davies at al., 2006) ................................................................. 63

Figure 3-17: Lateral Wall Deflection Measurements for Excavation Level 10 (Davies at al., 2006) ...................................................................................................................... 64
| Figure 3-18: | Summary of S335 Strut Load Data vs. Date & Time (Whittle, 2006) | 65 |
| Figure 3-19: | Undrained Strength Profiles used in FE simulations for Type M3 Excavation Support System (Whittle & Davies, 2006) | 66 |
| Figure 3-20: | Effect of Undrained Shear Strength Profile on Wall Deflections for Type M3 Excavation Support System (Whittle & Davies, 2006) | 67 |
| Figure 3-21: | Effect of the Analysis Method and Undrained Shear Strength Profile on Computed Strut Loads (Whittle & Davies, 2006) | 68 |
| Figure 4-1: | (a) Typical Irreversible Stress-Strain Response, and (b) Typical Modulus Variation for Soil (Muir Wood, 2004) | 81 |
| Figure 4-2: | Elastic-Perfectly Plastic Model: (a) Stress-Strain Response & (b) Modulus Variation (Muir Wood, 2004) | 82 |
| Figure 4-3: | Mohr-Coulomb Model Representation for a Drained Triaxial Shear Test | 83 |
| Figure 4-4: | Effective stress path for undrained plane strain shearing using EPP (Mohr-Coulomb) soil model | 84 |
| Figure 4-5: | CSSM (Critical State Soil Mechanics) and MCC (Modified Cam Clay) | 85 |
| Figure 4-6: | Conceptual model of Unload-Reload used by MIT-E3 for Hydrostatic Compression: (a) Perfect Hysteresis, (b) Hysteresis & bounding Surface Plasticity (Whittle & Kavvadas, 1994) | 86 |
| Figure 4-7: | Yield, Failure & Load Surfaces used in MIT-E3 Model (Whittle & Kavvadas, 1994) | 87 |
| Figure 4-8: | Evaluation of Model Input Parameters C, n for Hydrostatic Swelling (Whittle & Kavvadas, 1994) | 88 |
| Figure 4-9: | Effect of Model Parameters St and c on Prediction of Effective Stress Paths for K0-Normally Consolidated Clay in Undrained Triaxial Compression and Extension Tests (Whittle & Kavvadas, 1994) | 89 |
| Figure 4-10: | Compression and swelling properties of the Upper and Lower Marine Clays | 91 |
Figure 4-11: 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.

Figure 5-1a: North Wall Section Stratigraphy at M3 area (COI, 2005).

Figure 5-2: Post Collapse Site Exploration for S333, S335 & S339 Sections showing Boreholes (Arup 2005).

Figure 5-3: Extension of the F1 Fluvial Sand at the M3 Type Area (Arup, 2005).

Figure 5-4: S335 Section geometry used in FE model (Corral and Whittle, 2010).

Figure 5-5: (a) Ground Water Heads for Initial Conditions, after Applying Initial Drained Equilibrium: (b) South Section and (c) North Section.

Figure 5-6: Comparison of in situ stresses and undrained strengths of marine clay used in FE model (Corral & Whittle, 2010).

Figure 6-1: S335 Wall Deflection Comparison between MC and MIT-E3: (a) South Wall (b) North Wall.

Figure 6-2: Maximum Lateral Wall Deflections vs. Elevations.

Figure 6-3: Measured and Predicted Wall Deflections for Excavation Levels 5 and 6.

Figure 6-4: Measured and Predicted Wall Deflections for Excavation Levels 7 and 8.

Figure 6-5: Measured and Predicted Wall Deflections for Excavation Levels 9 and 10.

Figure 6-6: Computed and Measured Maximum Wall Deflections (a) South Wall (b) North Wall.

Figure 6-7: Bending Moments for (a) South Wall, and (b) North Wall.

Figure 6-8: S335-Strut Loads for Strut Levels 1, 2 & 3, using MC.

Figure 6-9: S335-Strut Loads for Strut Levels 1, 2 & 3, using MIT-E3.

Figure 6-10: S335-Strut Loads for Strut Levels 4, 5 & 6, using MC.
Figure 6-11:  S335-Strut Loads for Strut Levels 4, 5 & 6, using MIT-E3 ............................... 127

Figure 6-12:  S335-Strut Loads for Strut Levels 7, 8 & 9, using MC ................................. 128

Figure 6-13:  S335-Strut Loads for Strut Levels 7, 8 & 9, using MIT-E3 ............................... 129

Figure 6-14:  Comparison of Measured and Predicted Maximum Strut Loads....................... 131

Figure 6-15:  Predicted Surface Vertical Settlements............................................................... 132
LIST OF TABLES

Table 2-1: The Various Stages of the MRT Circle Line Project (COI, 2005) .................. 26

Table 2-2: The Various Sections of C824 and Sections of Cut & Cover 2: CC2 (COI, 2005) .................................................................................................................. 32

Table 2-3: Definition of Methods A, B, C & D (COI, 2005) ............................................. 37

Table 2-4: Comparative Study of Method A & Method B for Strut Loads at Type M3 (COI, 2005) .............................................................................................................. 39

Table 4-1: Input Parameters for MIT-E3 Constitutive Soil Model: Upper Marine Clay (UMC) and Lower Marine Clay (LMC) ................................................................. 90

Table 4-2: MC Model Parameters for Soil Layers used in Current Analyses (S335 Section) 93

Table 5-1: MC Model parameters for soil layers at S335 model. ....................................... 103

Table 5-2: (a) JGP Raft Elevations, (b) Elevations of Type Walls, and (c) Wall Material Properties (COI, 2005 and Arup, 2005). ................................................................. 104

Table 5-3: Strut Properties & Pre-Load Struts Assumed, and Elevation (Reduced Level) for each Strut Level (Arup, 2005) ................................................................. 105

Table 5-4: Calculation Phases used in FE simulations for S335 Section ......................... 108

Table 6-1: Summary of Computed Maximum Lateral Wall Deflections & Corresponding Elevations ......................................................................................... 115

Table 6-2: Summary of Computed and Measured Maximum Lateral Wall .............................................. 120

Table 6-3: Summary of Maximum Wall Bending Moments .............................................. 123

Table 6-4: S335 Predicted Maximum Strut Loads ............................................................. 130

Table 6-5: Predicted Maximum Surface Vertical Settlements ........................................... 133
1 INTRODUCTION

A 30 m deep excavation in marine clay next to Nicoll Highway in Singapore collapsed at 3.30 pm on 20th April 2004. The excavation support system comprised 41.3 m deep diaphragm wall panels with 10 levels of preloaded cross-lot struts and two rafts of jet grout piles (JGP). Failure occurred when excavations progressed below the 9th level of strutting when the upper JGP (sacrificial) was removed. Figure 1-1 shows a picture taken after the collapse.

This thesis re-analyzes the collapse of the excavation using the generalized effective stress soil model, MIT-E3 (Whittle, 1987). This work enabled by a program of laboratory tests that were conducted as part of the post-collapse investigations at the site.

The thesis begins by reviewing the original project, corresponding to the first phase of construction for the new Circle Line Subway in Singapore, contract C-824. Chapter 2 describes the event leading up to the collapse and the causes of the collapse as determined by the Committee of Inquiry in May 2005 (COI, 2005).

Chapter 3 describes the local geology site conditions and soil properties together with key observations from data as reported by Davies et al., 2006. The chapter also discusses errors in the original design of the support system associated with the modeling of the undrained shear strength profile (based on Whittle & Davies, 2006).
Chapter 4 summarized the key features of the MIT-E3 soil model and describes model calibration for the Singapore Marine Clays at the site. This is made possible using laboratory data obtained from the post-collapse site investigation (Kiso-Jiban, 2005; unpublished).

Chapter 5 describes the 2-D finite element modeling for the instrumented section of the project at the strut line S335. Modeling is carried out using commercial Plaxis™ (v.8.5). This is the first application of Plaxis that includes MIT-E3. The model was integrated within the Plaxis program by Akl and Bonnier (pers. comm., 2008).

Chapter 6 analyses all results obtained from the simulations. The effects of soil modeling on predictions of lateral wall deflections, bending moments, strut loads & vertical settlements are discussed in detail, comparing results from MIT-E3 with those obtained by conventional models. The predictions are then compared with monitoring data at S335.

Chapter 7 presents the summary, conclusions, and recommendations from this work.
Figure 1-1: Nicoll Highway Collapse – April 20th 2004 (COI, 2005)
2 PROJECT & COLLAPSE REVIEW

2.1. Overview

The new Circle Line (CCL) project in Singapore was intended to improve time for commuters around the downtown district, by passing busy interchanges such as City Hall and Raffles Place Figures 2-1, 2-2. The CCL project consists of twin subway tunnels 33.6 km long with 26 operating stations, and provision for 3 future stations. The project had an estimated budget of S$6.7 billion and was originally scheduled for completion in 2009.

Figure 2-2 and Table 2-1 shows that the project was to be built in 5 main stages. The first phase CCL1 has a total length of 5.4 km and was divided contracts C824 and C825. This thesis focuses on excavations for contract C824 between the proposed Nicoll Highway Station east to Kallang River, Figure 2-3. These locations are shown in more detail in Figure 2-5.

A complete description of the project can be found at the “Report of the Committee of Inquiry into the incident at the MRT Circle Line Worksite that led to The Collapse of Nicoll Highway on April 20th 2004 (COI, 2005).

2.2. Contract C-824

Contract C824 involved approximately 2.8 km of route and included the construction of Nicoll Highway and Boulevard Stations, and the linking tunnels. The original contract period ran from
May 30th 2001 to Jan 30th 2006. This design-build contract was awarded to a joint venture of Nishimatsu Construction Company Limited and Lum Chang Building Contractors. All parties (Contractor and Sub-Contractors) involved in C824 are shown in Figure 2-4. Hereby, it can be seen who was the responsible for each design and construction part of the contract, including the design of permanent works, installation of the diaphragm wall, steel work and strutting, excavation, instrumentation, the jet grout piles – JGP (Figure 2-6), the bored piles, the strut monitoring works, and, the soil investigation works. Table 2-2 outlines the different sections involved in C824, describing the type of sections, the wall type or wall label, and the approximate length in meters. The Collapse occurred in the Cut & Cover Tunnel 2 (CC2) area in design section M3 adjacent to the Temporary Access Shaft.

Figure 2-7 shows the design cross-section for the (intended) 33.3 m deep excavation comprising 0.8 m thick diaphragm wall panels that extend through deep layers of Estuarine and Marine clays (Kallang formation) and are embedded a minimum of 3 m 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.

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 2-8. 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 – 2.0 m outside the South wall.

It is extremely important to denote that because of the length of the CC2 section, variation of ground conditions, among other singularities of the area, the project demanded different widths of excavation, depths of diaphragm walls, numbers of levels of struts and JGP thicknesses. In order to overcome this complexity, NLC subdivided the temporary work into different types, as Table 2-3.

2.3. Events Leading Up the Collapse

From March 2003 to April 2004, the construction of the temporary retaining wall system for C824 was surrounded by multiple problems. These were detailed by the COI report (COI, 2005) and included incidents at Launch Shaft 2, and cut and cover excavations Type K, and Type M2 & Type M3 are included.

Problems in the collapse area begin when measured lateral wall deflection exceeded the design level on February 23th 2004 (145 mm) when excavation was progressed to a depth of 18.3 m (corresponding to 84.6 m RL in Figure 2-7). The contractor presented backanalyses (BA1) on March 5th 2004 to increase allowable design wall deflection, from 145 mm to 253 mm.
The revised levels for wall deflection were exceeded on March 30th 2004 (excavations at 78.3 m RL). This provided further re-analyses of M3 (BA2) that were finally approved on April 19th which design levels increased to 359 mm. These design levels were exceeded the following day at M3 (as measured by one of the inclinometer on the south side of the wall). On April 20th, the first signs of collapse were related to observations of failure in the 9th level strut-waler connections (these were observed over a period of 4 hours, 9 am-1 pm). Strut loads measured at S335 response at about 11am when the load drops in the 9th level strut increasing by an equal amount in the 8th level strut (indicating a transfer of load upwards trough the bracing system). Contingency actions at the site, including adding concrete to strengthen strut-waler connections were ineffectual and failure at the 8th level strut-waler connection began at 3:00 pm. The excavation collapsed catastrophically at 3:30 pm with the tragic loss of 4 lives. Further details can be found on the COI report (COI, 2005).

2.4. Collapse Causes Summary

Based on Committee of Inquiry Report (COI, 2005), the main causes of the collapse can be attributed to two critical design errors in the temporary retaining wall system. These two errors correspond to:

1. The under-design of the diaphragm wall using Method A.

2. The under-design of the waler connection:
   a. Incorrect estimations of loads on waler connections for different strut conditions.
b. Replacement of plate waler connections by C-channels which do not possess ductile behavior.

Figure 2-9 presents a more detailed breakdown of causality as proposed by Arup (2005).

From a geotechnical perspective, under-design of the diaphragm wall using Method A is one of the key factors. The terminology Method A refers to the particular use of the Plaxis program to present undrained behavior of low permeability clays using the reference Mohr-Coulomb (MC) model. Table 2-3 shows that there are three possible methods for representing undrained behavior using the MC model.

Method A refers to the approach where effective stress strength parameters \((c', \phi')\) are used as inputs. In this case, undrained shear strength is obtained implicitly by the MC model (based on assumptions of linearly-elastic, perfectly-plastic material behavior. Whittle & Davies (2006) have shown that using Method A (with parameters observed from drained strength of Singapore Marine Clay) greatly overestimates the undrained shear strength of the profile. As a result, the original design underestimated the wall deflections (Figure 2-11) and bending moments in the diaphragm used at section M3 (by a factor of 2). It also led the designers to undersize JGP rafts (see Figure 2-9). However, as shown in Table 2-4, Method B has a much smaller effect on predictions of strut loads. Indeed only 2 levels of struts are expected to carry higher loads according to Method B (levels 6 and 9).
In comparison of the experts to the COI all used Method B (Table 2-3), which uses the undrained strength profile directly as input to the MC model (c'→su, φ=0; i.e. total stress strength parameters are used). This method is considered reliable assuming there is minimal consolidation occurring in the clay layer over the time frame of the construction.

Figure 2-10 summarizes the stress paths for each of the 4 methods in q-p’ space (where q=(σ1-σ3) is the shear stress and p’=1/3(σ′1+σ′2+σ′3) is the mean effective stress). This example assumes initial hydrostatic conditions (i.e. K₀=1), showing the value of undrained shear strength reached using each method. As may be noted in this Figure, Method A dramatically overestimated the su (or cu) value.
Table 2-1: The Various Stages of the MRT Circle Line Project (COI, 2005)

<table>
<thead>
<tr>
<th>CCL Stage</th>
<th>Description</th>
<th>No of Stations</th>
<th>Stage Length (km)</th>
<th>Year Start / Expected Year of Completion</th>
</tr>
</thead>
<tbody>
<tr>
<td>CCL1</td>
<td>From Dhoby Ghaut to Boulevard station</td>
<td>6</td>
<td>5.4</td>
<td>2001 / 2006</td>
</tr>
<tr>
<td>CCL2</td>
<td>From Old Airport Road to Upper Paya Lebar Station</td>
<td>5</td>
<td>5.5</td>
<td>2002 / 2006</td>
</tr>
<tr>
<td>CCL 3</td>
<td>From Bartley to Marymount Station</td>
<td>5</td>
<td>5.7</td>
<td>2003 / 2008</td>
</tr>
<tr>
<td>CCL 4</td>
<td>From Thomson to NUH station</td>
<td>8</td>
<td>10.4</td>
<td>2004 / 2009</td>
</tr>
<tr>
<td>CCL 5</td>
<td>From West Coast to HarbourFront station</td>
<td>5</td>
<td>6.6</td>
<td>2004 / 2009</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>29 stations</strong></td>
<td></td>
<td><strong>33.6</strong></td>
<td></td>
</tr>
</tbody>
</table>
Figure 2-1: Subway system in Singapore, showing new Circle Line project in 2006
Figure 2-2: Overview of Circle Line Stages (COI, 2005)
Figure 2-3: Overview of Circle Line Stage 1: CCL1 (COI, 2005)
Figure 2-4: Chart showing the Relationship of Parties Involved in C824 (COI, 2005)
Figure 2-5: Contract C824 involved approximately 2.8 km of route and included the construction of Nicoll Highway & Boulevard Stations, and the linking tunnels (Davies et al., 2006)
### Table 2-2: The Various Sections of C824 and Sections of Cut & Cover 2: CC2 (COI, 2005)

<table>
<thead>
<tr>
<th>Section of C824</th>
<th>Wall Type</th>
<th>Approx. Route Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cut &amp; Cover Tunnel 1 (CC1)</td>
<td>Type A, B, B1 and C</td>
<td>372</td>
</tr>
<tr>
<td>Nicoll Highway Station (NCH)</td>
<td>Type D2, D1A, E1, F1, E2, D1, E3, E4, F2, G1 and G2</td>
<td>327</td>
</tr>
<tr>
<td>Cross Over Box (XOB)</td>
<td>Type H, I, J and K</td>
<td>199</td>
</tr>
<tr>
<td>Cut &amp; Cover Tunnel 2 (CC2)</td>
<td>Type L, M1, M2 and M3</td>
<td>211</td>
</tr>
<tr>
<td>TSA Shaft (TSA)</td>
<td>Type N</td>
<td>35</td>
</tr>
<tr>
<td>Bored Tunnels (under Kallang Basin)</td>
<td>-</td>
<td>799</td>
</tr>
<tr>
<td>Launch Shaft 1</td>
<td>Type Q1 and Q2</td>
<td>33</td>
</tr>
<tr>
<td>Boulevard Station (BLV)</td>
<td>Type R1, S and R2</td>
<td>246</td>
</tr>
<tr>
<td>Boulevard Siding (BLS)</td>
<td>Type T, U1, U2, V, W1, W1a, W1b, W2, W2a and Y</td>
<td>475</td>
</tr>
<tr>
<td>Bored Tunnels (over-run after BLS)</td>
<td>-</td>
<td>71</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td><strong>2768</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Section of CC2</th>
<th>Approx. Route Length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type L</td>
<td>46</td>
</tr>
<tr>
<td>Type M1</td>
<td>44</td>
</tr>
<tr>
<td>Type M2</td>
<td>88</td>
</tr>
<tr>
<td>Type M3</td>
<td>33</td>
</tr>
<tr>
<td><strong>Total Route Length</strong></td>
<td><strong>211</strong></td>
</tr>
</tbody>
</table>
Horizontal Slab Construction

Triple Rod Jet Grouting – Construction Individual Elements

Horizontal Slab Showing Inter Locking Elements

Figure 2-6: Construction Principle of the Jet Group Pile – JGP – Slab (COI, 2005)
Figure 2-7: M3 typical design cross-section of excavation support system (Whittle and Davies, 2006)
Figure 2-8: Plan showing the structural support system and 9th level strutting and monitoring instrumentation (Corral & Whittle, 2010)
Figure 2-9: Diagram: Collapse Causes interpreted by ARUP (ARUP, 2005)
Table 2-3: Definition of Methods A, B, C & D (COI, 2005)

<table>
<thead>
<tr>
<th>Method</th>
<th>Plaxis Material setting</th>
<th>Material Model</th>
<th>Parameters</th>
<th>Computed stresses</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Undrained</td>
<td>Mohr-Coulomb</td>
<td>$c', \phi'$ (effective)</td>
<td>Effective stress and pore pressure</td>
</tr>
<tr>
<td>B</td>
<td>Undrained</td>
<td>Mohr-Coulomb</td>
<td>$c_u, \phi_u$ (total)</td>
<td>$E', v'$ (effective)</td>
</tr>
<tr>
<td>C</td>
<td>Non-porous</td>
<td>Mohr-Coulomb</td>
<td>$c_u, \phi_u$ (total)</td>
<td>$E_u, v_u=0.495$ (total)</td>
</tr>
<tr>
<td>D</td>
<td></td>
<td></td>
<td></td>
<td>As in Method A, for other soil models</td>
</tr>
</tbody>
</table>
Figure 2-10: Mohr-Coulomb Failure Model: Undrained Shear Strengths derived from Methods A, B, C & D used in the C824 Project (COI, 2005)
## Table 2-4: Comparative Study of Method A & Method B for Strut Loads at Type M3 (COI, 2005)

<table>
<thead>
<tr>
<th>Strut Row</th>
<th>Predicted Strut Load Using Method B (kN/m)</th>
<th>Design Strut Load (unfactored) Using Method A (kN/m)</th>
<th>Ratio Method B to Design Strut Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>379</td>
<td>568</td>
<td>67%</td>
</tr>
<tr>
<td>2</td>
<td>991</td>
<td>1018</td>
<td>97%</td>
</tr>
<tr>
<td>3</td>
<td>1615</td>
<td>1816</td>
<td>89%</td>
</tr>
<tr>
<td>4</td>
<td>1606</td>
<td>1635</td>
<td>98%</td>
</tr>
<tr>
<td>5</td>
<td>1446</td>
<td>1458</td>
<td>99%</td>
</tr>
<tr>
<td>6</td>
<td>1418</td>
<td>1322</td>
<td>107%</td>
</tr>
<tr>
<td>7</td>
<td>1581</td>
<td>2130</td>
<td>74%</td>
</tr>
<tr>
<td>8</td>
<td>1578</td>
<td>2632</td>
<td>60%</td>
</tr>
<tr>
<td>9</td>
<td>2383</td>
<td>2173</td>
<td>110%</td>
</tr>
</tbody>
</table>
Figure 2-11: Lateral Wall Deflection comparison between Method A & B (COI, 2005)
3 SITE CONDITIONS

3.1. Geology and Site Conditions

Figure 3-1 shows a generalized map of surficial geology for Singapore, illustrating the main geological formations which can be found in the island. Circle Line contract C824 is located within underlying by the Kallang formation and affected by more recent land reclamation (dating from 1940’s; see Figure 3-4). The M3 section comprises more than 40m of marine sediments underlying by variable deposits of Old Alluvium.

Chiam et al. (2003) and Pitts (1984) state that the Old Alluvium (OA) is known to be the oldest of the drift deposits and is mainly found in the eastern and northwestern parts of Singapore. It is an extension of early Pleistocene deposits found in southern Johore and exists as a virtually uninterrupted sheet either at the surface or buried beneath younger deposits. Davies (1984) shows typical conditions in the Kallang formation where the marine deposits are formed in valley cut into the Old Alluvium, Figure 3-3. This is consistent with the alluvial drainage pattern outlined in a review of the Pleistocene deposits by Gupta et al. (1987).

More recently, Mote et al. (2009) have discussed the existence of Palaeochannels locally in the Old Alluvium. Figure 3-2 shows their statistical interpretation of meandering channels and their likely occurrence in the vicinity of C824.
Regarding the Kallang Formation, the main two units of marine clay (upper and lower) are found consistently. It is believed that the upper clay was deposited during the Holocene era, but the lower one was deposited more than 10,000 years ago (Hanzawa and Adachi 1983; Pitts 1984).

The presence of land reclamations plays a very important role to comprehend the state of the main soil layers in some areas of Singapore. Two periods of reclamation have been identified. The first in the 1930-1940’s and the second in the 1970’s, as shown in Figure 3-4. The extent of the second reclamation is clearly illustrated by a 1969 aerial photo of the site, Figure 3-5.

Whittle and Davies (2006) present an illustration (Figure 2-7) of the original design of the lateral earth support system (based on one borehole – ABH32) at the initiation of the collapse area (type M3). The cut-and-cover section is approximately 20m wide, with a final formation depth of 33 m. Most of the diaphragm walls (i.e. excavation support system) have a thickness of 80 cm and a varying embedment in the Old Alluvium (OA). The walls were designed with 10 levels of pre-loaded cross-load bracing and two rafts of jet grout piles. The collapse occurred (April 20th 2004) following the installation of strut level 9 and removal of the upper sacrificial JGP raft when the formation was at approximately 72.3 m RL. No level 10 struts were installed.

The location of the pre-tender and post-tender boreholes, piezocones and diaphragm wall panels in the M3 section are presented in Figure 3-6.

The typical soil profile comprises (Figure 2-7) Fill, Upper Estuarine, Upper Marine Clay (UMC), transition clay unit F2, Lower Marine Clay (LMC), Estuarine units, F2 Clay and Old Alluvium (OA). Whittle and Davies (2006), indicate that for practical purposes (e.g., estimation of
undrained shear strength) there is little to distinguish the lower Estuarine and Marine clay units of the Kallang formation (both have plasticity indices, $I_p = 35-55\%$, while the Estuarine has a slightly higher liquid limit, $w_L = 70-100\%$ than LMC, $w_L = 65-80\%$). The thicknesses of each layer vary considerably from one point to another. Figures 3-7 and 3-8 show two sections constructed using data from the pre-tender borings. The bases of the marine clay/estuarine deposits are clearly defined and show a significant change in elevation across the site. The deepest sections of Marine clay are in M3 and in the south side of the excavation. The top of the buried layer of OA is more difficult to interpret from the complex borehole descriptions. Whittle and Davies (2006) present the elevation of the bottom of LMC and the top of OA based on a definition $N_{spt}>30$, Figures 3-9 and 3-10.

Contour plots assembled for the COI (Figure 3-10) show clearly a valley within the OA. The valley trends NNE-SSW and is deepest between the TA Shaft and the instrumented strut line S335.

The OA classified as very dense silty sands with transitioning with depth to very stiff to hard, silty clay. Also, it is convenient to separate the OA in different sub-layers depending on the SPT blowcounts (Whittle & Davies, 2006). There is evidence that, at the North side, transitional fluvial sand (F1) exists between the OA and the Marine Clay, and, fluvial clays (F2) at the South side.

The groundwater table in the fill varies from 100 to 100.5 m (RL). However, standpipe piezometer data from boreholes indicate an excess of piezometric head (estimated at $H \approx 103$ m) below the Marine Clay (COI, 2005; Figure 3-11). Surface settlements measured prior to
construction (2000-2001), Figure 3-12. Hence, Whittle and Davies (2006) conclude that there is on-going consolidation within the Marine Clay that can affect excavation performance.

The most reliable data for estimating the undrained shear strength was provided by piezocone penetration records (Whittle & Davies, 2006). Figure 3-13 shows a comparison of the undrained shear strength captured by four piezocone records. These are interpreted using a cone factor \( N_{kt} = 14 \). The results are compared with a design strength line for normally consolidated marine clay using an average undrained strength ratio \( \frac{s_u}{c}\sigma^\prime_v = 0.21 \). (This corresponds to the assumed undrained properties in the original Geotechnical Interpretative Memorandum; GIM, 2001). 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.

3.2. Construction

The construction sequence of the cut-and-cover excavation at C824 has the following main stages: 1) install diaphragm walls, 2) install bored piles, 3) drive king posts, 4) install jet gout piles (JGP), 5) excavation and the strut pre-loading (10 stages), and then 6) permanent works (tunnel supports). Collapse occurred prior to installation of the 10th level of strutting.
According to Whittle and Davies (2006), the original design idea was to achieve 3m of embedment of the diaphragm wall panels within the Old Alluvium. However, the construction records show that the individual panels of the diaphragm wall were actually installed to specified design elevations rather than embedment requirements. Figure 3-15 show the as-built wall panels on the North and South side of the excavation. Panel embedment was much less than 3m for a series of panels close to the TSA Shaft where the Old Alluvium is at the greatest depth. At the instrumented section S335, the embedment of panels M307 and M302 is critically quite close to design (1.8 – 2.6 m). Figure 3-15 also shows the gap in the diaphragm wall due to the presence of the 66 kv cable. The designers increased panel with and reinforcement of the adjacent panels (M303-M304, M212-M306) to enable arching of earth pressures across these gaps.

The JGP was designed as two relatively thin rafts in the M3 area (upper 1.6 m and lower 2.5 m thick, respectively). Jet group piles were installed using a double fluid jetting system with parameters calibrated for conditions in the upper Marine clay (i.e. jetting parameters were selected to achieve required column diameter). The design shear strength of the JGP rafts was $s_u=300$ kPa based on prior empirical data. Figure 3-14 shows the sequence of column installation that was intended to produce an integral raft of soilcrete. In practice, records on actual pile installation were incomplete and do not enable validation of the actual construction. Whittle and Davies (2006) note that is unlikely that the design column dimensions were achieved for JPG piles installed trough higher strength OA or F2 layer.

### 3.3. Instrumentation and Monitoring Data

Instrumentation is critical for monitoring and controlling the excavation performance.

Page | 45
There are two inclinometers available for interpreting wall deflections at the M3 section, I-104 (south side 2m behind the wall) and the I-65 (north side within diaphragm wall), Figure 2-8.

The loads in each strut level at the line S335 (Figure 2-8) were also monitored using load cells (recording strut pre-load) and sets of three strain gauges.

Davies et al. (2006) present the lateral wall deflection measurements for excavation levels 6, 7, 8, & 9, Figure 3-16 and for conditions immediately prior to the failure, April 17-20, Figure 3-17. The data shows very similar measured deflections of the north and south trough early March, 2004 (strut level 7 installed). Thereafter, the south wall countries to deform significantly below the formation level, reading 300 mm at level 9 (April 5), while there are negligible movements at the North wall over this same time period (March-April; levels 7-9) This asymmetry is closely linked to the underlying stratigraphy.

Figure 3-18 shows a summary of S335 strut load data for levels 5 trough 9. These data were validated by the experts at the COI and show an unexplained disconnect between the imposed pre-load (measured by load cell) and subsequent strut loads obtained by strain gauges. The data show a consistent pattern of increased load in the succeeding excavation step (after pre-load), a decrement due to installation of the next strut, and general reduction in load thereafter.

3.4. Effect of Undrained Strength Profile on Predicted Performance

Whittle and Davies (2006) carried out a series of finite element simulations in order to investigate the role of the analysis method (A vs. B) and selected undrained shear strength
profiles on performance of the M3 excavation. Figure 3-19 shows 4 profiles used in these calculations:

1. A[NLC] is a calculation that reproduces the original design using Method A. The undrained shear strength of the Marine and Estuarine Clay units are based on the use of effective stress strength parameters ($c'$, $\phi'$) provided by GIM (2001).

2. B[GIM] uses Method B ($s_u$, $\phi' = 0^\circ$) to represent the undrained strength profile for the Marine and Estuarine clay layers with undrained shear strengths specified by GIM (2001).

3. GIM* also uses Method B, but makes three amendments that are consistent with more detailed interpretation of local ground conditions. These relate to undrained strength of the Old Alluvium and as-constructed thickness of JGP rafts.


The resulting predictions of lateral wall deflection results are summarized in Figure 3-20 at level 7 excavation and at the final dig level. Method A clearly underestimates the magnitudes of wall deflections by more than a factor of 2 compared with the tree method analyses. Additionally, the maximum strut loads at each excavation level 6, 7 & 9, and for each $s_u$ profile are presented in Figure 3-21. These results show that all 4 sets of analyses predict overloading of the as-built strut-waler connection at level 9.
Figure 3-1: Geology of Singapore showing the North East Line Location (Shirlaw et al., 2000)
Figure 3-2: Results of recent studies of paleo-channels beneath Kallang formation (a) Transition from Bedrock Controlled Channels into Meandering Channels (b) Location of Confirmed Palaeochannels, (c) Palaeochannels Likelihood Mapping (Mote et al. 2009)
Figure 3-3: Typical Profile in Kallang Formation Area (Davies, 1984)
Figure 3-4: Land Reclamations in the 30's & 70's (Davies at al., 2006)
Figure 3-5: Prior Reclamation of Land: Aerial Photo taken 5th April 1969 (Davies et al., 2006)
Figure 3-6: Location of Boreholes, Piezocones & Diaphragm Wall Panels in the M3 section. Blue and red circles and triangles represent the boreholes and piezocones for pre-tender and post-tender, accordingly. (Whittle & Davies, 2006)
Figure 3-7: Cross Section A-A, showing the Required Final Depth of the Excavation and Variations of OA & LMC (Davies et al. 2006)
Figure 3-8: Cross Section B-B, showing the Required Final Depth of the Excavation and Variations of OA & LMC (Davies et al. 2006)
Figure 3-9: Contours of base of Lower Marine Clay, in m, RL (Whittle & Davies, 2006)
Figure 3-10: Contours of top of Old Alluvium (for $N_{spt}>30$), in m, RL (Whittle & Davies (2006))
Figure 3-11: Locations of Piezometer & Settlement Points and Reclamation History (COI, 2005)
Figure 3-12: Ground Surface Settlement Measured prior to Construction (Whittle & Davies, 2006)
Figure 3-13: Undrained shear strength profiles (Corral & Whittle, 2010)
Figure 3-14: Construction Sequence for JGP slabs (COI, 2005)
Figure 3-15: Summary of As-Built South & North Diaphragm Wall Panel Embedment in M3 area (Whittle & Davies, 2006)
Figure 3-16: Lateral Wall Deflection Measurements for Excavation Levels 6, 7, 8, & 9 (Davies et al., 2006)
Figure 3-17: Lateral Wall Deflection Measurements for Excavation Level 10 (Davies et al., 2006)
Figure 3-18: Summary of S335 Strut Load Data vs. Date & Time (Whittle, 2006)
Figure 3-19: Undrained Strength Profiles used in FE simulations for Type M3 Excavation Support System (Whittle & Davies, 2006)
Figure 3-20: Effect of Undrained Shear Strength Profile on Wall Deflections for Type M3 Excavation Support System (Whittle & Davies, 2006)
Figure 3-21: Effect of the Analysis Method and Undrained Shear Strength Profile on Computed Strut Loads (Whittle & Davies, 2006)
4 CONSTITUTIVE BEHAVIOR OF KALLANG FORMATION SOILS

4.1. Soil Behavior and Generalized Effective Stress Soil Models

Figure 4-1 illustrates the typical features of the shear-stress-strength properties of clays. The following key points should be noted:

1. The stress-strain properties are non-linear; there is no well defined range of linear behavior.
2. Shearing behavior first loading produces strains that are irrecoverable (i.e. not recovered during unloading).
3. To a first approximation, the unload-reload response is reversible and elastic (although typically non-linear and hysteretic).
4. The highest stiffness (tangent stiffness, Figure 4-1b) occurs immediately upon load reversal.

This behavior is described to a first order approximation by treating the soil as a linearly elastic-perfectly plastic material (EPP), Figure 4-2. The EPP model captures the key difference in stiffness between loading and unloading/reloading. However, it approximates the non-linear stress-strain behavior with an equivalent linear elastic stiffness, while the shear strength is simulated as a plastic flow at constant shear stress.

The EPP modeling of soil behavior is most commonly introduced by relating shear strength to the effective stress Mohr-Coulomb failure criterion, Figure 4-3.
The elastic-perfectly plastic Mohr-Coulomb (MC) soil model is fully defined by five input parameters that specify the linear elastic behavior (E, ν') and yield conditions (c', φ'). Perfectly plastic failure occurs when there is no volume change at failure and hence, the dilation angle ψ=0°.

Figure 4-4 shows the undrained effective stress path predicted by the MC for a saturated plane strain soil model. The initial K₀-stress condition is represented by A'. For the undrained shearing in the elastic range, the effective stress path is independent of changes in total stress and is constrained to a condition where Δp*=0 †† and hence A=0.5 †††. This is represented by the effective stress path A'-B' in Figure 4-4. Failure occurs at B' and hence, the undrained shear strength can be found from the initial stresses at the selected drained effective stress parameters (c', φ'). Thus:

\[ s_u = c' \cos \phi' + \frac{1}{2} (\sigma'_1 + \sigma'_3) \sin \phi' \] (4.1)

\[ \frac{s_u}{\sigma'_{vo}} = c' \cos \phi'/\sigma'_{vo} + \frac{1}{2} (1 + K_0) \sin \phi' \] (4.2)

Given these key results the MC model can be defined using either effective stress strength parameters (c', φ'; Method A) or using total stress strength parameters (c'=s_u, φ'=0; Method B). The key advantage of the MC model is the use of a small number of well-defined parameters for

† This is the geometric constraint for 2D FE models of excavation sections.
†† p' = 1/3 (σ'_{xx} + σ'_{yy} + σ'_{zz}) is the mean effective stress.
††† A = 1/3 corresponds to ESP for undrained triaxial shearing.
which there are extensive empirical data (e.g. GIM, 2001 for C824). As a result, MC is used as
the default model in the Plaxis™ program. Prior analyses of C824 performance have also relied
heavily on MC parameters to diagnosis the Collapse of Nicoll Highway due to lack of more
complete soil stress-strain-strength properties.

4.2. Generalized Effective Stress Soil Models

The goal of generalized effective stress soil models is to represent more reliably the effective
stress-strain-strength properties of soils. The models are calibrated to properties measured in
well-controlled laboratory element tests and must then be validated using results of more
complex tests on boundary value problems.

Most generalized models of clays are based on principles of Critical State Soil Mechanics
(CSSM; Schofield and Wroth, 1968) as embodied in Cam Clay models, most notable Modified
Cam Clay (MCC; Roscoe and Burland, 1968). The CSSM framework unifies observations on the
consolidation and shear behavior of reconstituted clays (under general drainage conditions). The
key features of the MCC model can be summarized as follows:

1. Conditions of yielding and large-strain (critical state) shear failure are de-coupled. Plastic
   yielding is described by a convex yield function, while a separate frictional strength when
   is used to represent critical state. Figure 4-5 shows that the critical state (CSL) is
described by a criterion:

\[ q_f = M p_f' \]  

(4-3)
where \( M = 6 \sin \varphi'_{\tau c} / (3 - \sin \varphi'_{\tau c}) \), \( \varphi'_{\tau c} \) is the frictional angle measured in conventional triaxial compression. \( q_f, p'_f \) are the shear stress and mean effective stress \(((\sigma'_1 - \sigma'_3), 1/3(\sigma'_1 + 2\sigma'_3))\) in triaxial space.

2. MCC assumes an elliptical yield function:

\[
f = q^2 - Mp'(p'_c - p')
\]  

where \( p'_c \) is the hydrostatic pre-consolidation pressure that defines the size of the surface, Figure 4-5.

3. \( p'_c \) is a hardening parameter that changes with plastic volume strain and enables the model to define the virgin consolidation and elastic swelling lines associated with consolidation (associated with compressibility parameters \( \lambda, \kappa \); respectively).

During undrained shearing (Figure 4-5), the effective stress path for a clay is controlled by associated plastic flow and hardening of the yield surface (elastic behavior occurs inside the yield function). Figure 4-5 illustrates the typical effective stress path for a normally consolidated clay (with \( K_0=1 \)). Undrained shearing produces positive shear-induced pore pressures (i.e. \( \Delta p'<0 \)). The undrained strength of the clay is controlled by hardening of the yield surface and the critical state friction parameter (i.e. by input parameters \( \lambda, \kappa \) and \( M \)).

Although the MCC model is very well known in geotechnical engineering it is not used so extensively in practice. This is due in part, to some key limitations of the modeling representing the elemental behavior of \( K_0 \)-consolidated clays:
1. It assumes that undrained shear strength is only mobilized at large strain critical state conditions. In contrast, there is extensive empirical data, especially for NC clays, that show the occurring at relatively small shear strains, and at stress ratios q/p' far from critical state.

2. It assumes isotropic behavior, such that the shear strength is independent of the direction of shearing. This is also contrary to the wealth of experimental data that find shear strength of natural of natural clays are strongly dependent on the direction of major principal stress at failure.

3. It assumes elastic behavior inside the yield surface and hence, does not describe accurately the observed non-linear and characteristic stress-strain properties of overconsolidated clays.

There has been extensive research to develop more reliable generalized effective stress soil models for describing the rate-independent behavior of normally to moderately overconsolidated clays. Whittle (1987) and Whittle and Kavvadas (1994) describe the formulation of MIT-E3. Figure 4-6 illustrates the conceptual framework for this model for the case of hydrostatic compression. The model comprises 3 concepts:

1. An elastoplastic model for normally consolidated clay that describes the anisotropic and strain-softening behavior (corresponding to behavior on the VCL, A, Figure 4-6).

2. Equations to describe the small strain non-linearity and hysteric response in unloading and reloading (A-B-A, Figure 4-6a).
3. Bounding surface plasticity for irrecoverable, anisotropic, and path-dependent behavior of overconsolidated clays (reload path B-C with irrecoverable stress $\Delta^p$, Figure 4-6b).

The anisotropic behavior of normally consolidated clays is described by introducing a generalized yield function with hardening parameters that control the size of the surface ($\alpha'$) and its orientation ($b$); Figure 4-7.

$$f = (s - \sigma'b) : (s - \sigma'b) - c^2\sigma'(2\alpha' - \sigma') = 0$$  \hfill (4-4)

where $c$ is the ratio of the semi-axes that defines the shape of the surface.

The model is able to describe the VCL using density hardening similar to MCC; while evolving anisotropic properties that are predicted through changes in orientation of the surface. The formulation assumes that the stress state of a $K_0$-normally consolidated soil is at the tip of the yield surface:

$$(\sigma', s) = 2\alpha'(1, b)$$  \hfill (4-5)

Anisotropic shear failure at large strains is modeled using a critical state failure criterion (Figure 4-7)

$$h = (s - \sigma'\xi) : (s - \sigma'\xi) - k^2\sigma'^2 = 0$$  \hfill (4-6)

where $\xi$ represents the orientation (anisotropy) tensor for critical state failure criterion.
The model formulations assumes that the orientation tensor is completely defined by both large strain friction angles measured and the triaxial compression and extension (φ'_c and φ'_e).

The unload-reload behavior in MIT-E3 is specified by a formulation that described a closed-symmetric hysteresis loop in the strain-stress response. The model requires the detection of a stress reversal point and functions to describe the non-linear stiffness relative to the reversal point. The volumetric response (Whittle and Kavvadas, 1994) is described by a tangent bulk modulus, K:

\[
K = \frac{1+e}{(1+\delta)\kappa_0} \sigma' \tag{4-7a}
\]

where \( \delta = Cn(\log e \xi + \xi_S)^{n-1} \) \tag{4-7b}

with \( \kappa_0, C, n \) constant parameters that are calibrated to measured data, while \( \xi \) and \( \xi_S \) are stresses defined relative to the reversal point. For hydrostatic swelling, \( \xi = \sigma_{rev}'/\sigma' \) is a measure of the overconsolidation ratio (defined in terms of mean stress)\(^\dagger\).

Figure 4-8 illustrates typical swelling behavior from the formulation for ranges of the parameters \( (C, n) \). The value of \( \kappa_0 \) is derived from small strain elastic properties (or alternatively from \( G_{\text{max}} \)).

Figure 4-9 shows typical predictions of the undrained shear behavior \( K_0 \)-normally consolidated clay using MIT-E3 (in triaxial compression and extension modes). The model is able to describe key features of the observed effective stress paths and shear stress strain behavior in these two

\(^\dagger\) \( \xi_S \) relates to changes in the stress ratio (\( \eta = S/\sigma' \))
laboratory element modes. In particular it is able to predict describe realistically the undrained strength anisotropic and characteristic stress-strain properties.

4.3. Calibration of MIT-E3 for Marine Clays

This section describes calibration of MIT-E3 model of units of the Upper and Lower Marine Clay at C824. This is made possible by a program of high quality laboratory tests carried out by Kiso-Jiban (unpublished) as part of the post-failure site investigation at the site of the Nicoll Highway Collapse.

The complete description of the MIT-E3 model is much more complex than MC and requires calibration of 15 input parameters. Of these parameters, 7 can be identified unambiguously following Whittle and Kavvadas (1994):

1. $K_{ONC}$ is the coefficient of lateral earth pressure (geostatic) normally consolidated clay that can be estimated from empirical formulas or measured during $K_0$-consolidation in either triaxial test or an oedometer with lateral stress measurements.

2. $\lambda$ and $e_o$ define the slope of the virgin consolidation line in $(e - \log_e \sigma')$ space, and can be obtained from an oedometer or CRS test.

3. $2G/K$ is the ratio of the tangential elastic shear modulus to the bulk modulus can be expressed in terms of the Poisson's ratio, as Equation (46). In addition, if the OCR at $K_0 = 1$ is known $(OCR)$, the ratio can be estimated from Equation (47).

$$\frac{2G}{K} = \frac{3(1-2v)}{(1+v)}$$

(4-8)
\[
\frac{2G}{k} = \frac{(1-K\text{ONC})OCR_1}{\frac{1}{3}(1+2K\text{ONC})OCR_1-1}
\] (4-9)

4. \( \kappa_0 \) is the elastic bulk modulus at small strains levels (i.e. \( K_{MAX} \) at stress reversal) should be estimated from the elastic shear wave velocity \( (G_{MAX}) \) measured either by lab resonant column tests or field investigations.

5. \( \phi'_{TC} \) and \( \phi'_{TE} \) are the friction angles for shearing at the critical state conditions in triaxial compression and extension modes of shearing (for practical proposes these friction angles can be estimated at 10% of axial strain from undrained triaxial shear tests).

The remaining eight input parameters must be determined by parametric studies (i.e. indirectly):

1. \( C, n \) define the non-linearity in the volumetric response for the perfectly hysteretic formulation are selected to match the swelling behavior in an oedometer or CRS test as shown in Figure (4-21).

2. \( \psi_0 \) is a dimensionless constant that controls the rate of change of anisotropy caused by the strain or stress history. The most useful lab tests are drained strain controlled tests (i.e. drained strain path tests).

3. \( S_t \) and \( c \) control the strain softening (principally in compression mode of shearing) and the undrained strength in the compression and extension modes of shearing are established from the undrained shear behavior of a \( K_0 \)-normally consolidated clay (preferably form Triaxial \( CK_0UC \) and \( CK_0UE \) tests).
4. \( \omega \) controls the non-linear behavior during undrained shearing at small strain levels and/or when the stress state is far from the bounding surface can be obtained from measurements of the secant shear modulus in the small strain range \( (\varepsilon_a = 0.001 - 0.05\%) \) from undrained triaxial compression tests on lightly OC clay \( (OCR = 1.5 - 2.0) \).

5. \( h \) defines the amount of residual plastic strain \( \Delta \) observed in hydrostatic unload-reload cycles.

6. \( \gamma \) represents the development of shear-induced pore pressures during undrained shearing of OC clays. For further explanation refer to the paper by Whittle & Kavvadas, 1994.

Table 4-1 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 (Figure 4-10) and \( K_0 \)-consolidated undrained triaxial shear tests (Figure 3-11) on specimens reconsolidated to the in situ stress conditions.

The compressibilities of the normally consolidated UMC and LMC units are well-characterized virgin consolidation lines with \( \lambda = 0.37 - 0.38 \), Figure 4-10a. The upper marine clay generally has higher in situ void ratio \( (e = 1.7 - 1.9) \) than the lower unit \( (e = 1.5 - 1.6) \). The marine clays show significant elastic rebound when unloaded. Figure 2b shows that recoverable axial strains, \( \Delta\varepsilon_a = 10-12\% \) 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, $k_0$, and then selected input values of $C$, $n$ (Table 4-1) to the swelling data as shown in Figure 4-10b.

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 4-11.

All of the specimens were consolidated to a common lateral stress ratio, $K_0 = 0.52$ prior to shearing. The measured data show a significant different in the average undrained triaxial compression strength ratios measured in these tests, $s_{uTC}/\sigma'_{ve} = 0.30$ 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 $c$, $S_0$, $\phi'_{TC}$, $\phi'_{TE}$, $\omega$ and $\gamma$ (Table 4-1). The remaining parameters in Table 4-1 have been estimated from prior studies on similar clays.
4.4. Parameters of Other Soil Units at C824

There are no direct laboratory data on the other soil units at S335. Hence, the current analyses use the MC model for these layers based on parameters specified in GIM (2001) or defined by Whittle and Davies (2006), see Table 4-2. These properties are used throughout the subsequent analyses.
Figure 4-1: (a) Typical Irreversible Stress-Strain Response, and (b) Typical Modulus Variation for Soil (Muir Wood, 2004)
shear stress

shear strain

shear strain

tangent

shear stiffness

initial loading

unloading

reloading

perfect plasticity

Figure 4-2: Elastic-Perfectly Plastic Model: (a) Stress-Strain Response & (b) Modulus Variation (Muir Wood, 2004)
Mohr-Coulomb Model Representation for a Drained Triaxial Shear Test

Notes:
1) Yield Function
\[ f = \frac{1}{2}(\sigma'_1 + \sigma'_s)\sin \phi' - \frac{1}{2}(\sigma'_1 - \sigma'_s) + c'\cos \phi' \]

2) \( \psi = f(\phi' - \phi'_{cv}) \), perfectly plastic case \( \psi = 0 \)

3) where \( \phi'_{cv} \) is the constant volume friction angle

Figure 4-3: Mohr-Coulomb Model Representation for a Drained Triaxial Shear Test
Figure 4-4: Effective stress path for undrained plane strain shearing using EPP (Mohr-Coulomb) soil model
Figure 4-5: CSSM (Critical State Soil Mechanics) and MCC (Modified Cam Clay)
Figure 4-6: Conceptual model of Unload-Reload used by MIT-E3 for Hydrostatic Compression: (a) Perfect Hysteresis, (b) Hysteresis & bounding Surface Plasticity (Whittle & Kavvadas, 1994)
Figure 4-7: Yield, Failure & Load Surfaces used in MIT-E3 Model (Whittle & Kavvadas, 1994)
Figure 4-8: Evaluation of Model Input Parameters C, n for Hydrostatic Swelling (Whittle & Kavvadas, 1994)
Figure 4-9: Effect of Model Parameters $S_t$ and $c$ on Prediction of Effective Stress Paths for $K_0$-Normally Consolidated Clay in Undrained Triaxial Compression and Extension Tests (Whittle & Kavvadas, 1994)
Table 4-1: Input Parameters for MIT-E3 Constitutive Soil Model: Upper Marine Clay (UMC) and Lower Marine Clay (LMC)

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Parameter</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</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>(Oedometer, CRS, etc.)</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>1-D Compressibility of virgin normally consolidated clay</td>
<td>10.0</td>
<td>9.0</td>
</tr>
<tr>
<td></td>
<td>$n$</td>
<td>Non-linear volumetric swelling behavior</td>
<td>1.50</td>
<td>1.50</td>
</tr>
<tr>
<td></td>
<td>$h$</td>
<td>Irreversible plastic strain</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>Ko-Oedometer or Ko-Triaxial</td>
<td>$K_{ONC}$</td>
<td>$K_o$ 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 unload)</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>Critical state friction angles in triaxial compression and extension (large strain failure criterion)</td>
<td>33.8°</td>
<td>27.1°</td>
</tr>
<tr>
<td></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=1: CKoUE</td>
<td>$S_t$</td>
<td>Amount of post-peak strain softening</td>
<td>3.0</td>
<td>5.0</td>
</tr>
<tr>
<td>OCR=2, CKoUC</td>
<td>$\omega$</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>
</tbody>
</table>
Figure 4-10: Compression and swelling properties of the Upper and Lower Marine Clays

a) Compression behavior

b) Swelling behavior
Figure 4-11: 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.
### Table 4-2: MC Model Parameters for Soil Layers used in Current Analyses (S335 Section)

<table>
<thead>
<tr>
<th>Layer</th>
<th>Reference Elevation RL, m</th>
<th>$su$ [c'] (kPa)</th>
<th>$\Delta su/\Delta z$ (kPa/m)</th>
<th>$\phi$ (°)</th>
<th>$E'$ (kPa)</th>
<th>$E'/\Delta z$ (kPa/m)</th>
<th>$v'$</th>
<th>$Ko$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill</td>
<td>-</td>
<td>[0]</td>
<td>-</td>
<td>30</td>
<td>10000</td>
<td>-</td>
<td>0.25</td>
<td>0.5</td>
</tr>
<tr>
<td>Upper MC</td>
<td>98.2</td>
<td>20</td>
<td>0.397</td>
<td>0</td>
<td>6913</td>
<td>137</td>
<td>0.25</td>
<td>0.52</td>
</tr>
<tr>
<td>F2 Clay</td>
<td>-</td>
<td>88</td>
<td>-</td>
<td>0</td>
<td>29250</td>
<td>-</td>
<td>0.25</td>
<td>0.7</td>
</tr>
<tr>
<td>Lower MC</td>
<td>83.4</td>
<td>31</td>
<td>0.79</td>
<td>0</td>
<td>10310</td>
<td>346</td>
<td>0.25</td>
<td>0.52</td>
</tr>
<tr>
<td>F2-2 Clay</td>
<td>-</td>
<td>88</td>
<td>-</td>
<td>0</td>
<td>29250</td>
<td>-</td>
<td>0.25</td>
<td>0.7</td>
</tr>
<tr>
<td>F1-Sand</td>
<td>-</td>
<td>[0]</td>
<td>-</td>
<td>30</td>
<td>10000</td>
<td>-</td>
<td>0.25</td>
<td>0.5</td>
</tr>
<tr>
<td>OA-weathered</td>
<td>61.6</td>
<td>100</td>
<td>52</td>
<td>0</td>
<td>33250</td>
<td>13600</td>
<td>0.25</td>
<td>1</td>
</tr>
<tr>
<td>OA-competent</td>
<td>-</td>
<td>500</td>
<td>-</td>
<td>0</td>
<td>167500</td>
<td>-</td>
<td>0.25</td>
<td>1</td>
</tr>
<tr>
<td>JGP</td>
<td>-</td>
<td>300</td>
<td>-</td>
<td>0</td>
<td>250000</td>
<td>-</td>
<td>0.15</td>
<td>-</td>
</tr>
</tbody>
</table>

† Linear variations with depth in stiffness and shear strength within soil layers starting at reference elevation (constant values above)
5 FINITE ELEMENT MODELING

5.1. Model Geometry

The cross-sectional geometry at S335 has been developed using data from the original site investigation and the post-collapse investigation.

Figure 5-1a and 5-1b show the detailed soil profiles along the north and south walls in the M3 design area. Figure 5-2 shows the location of all boreholes available at the site. The figure locates boreholes that have been used to develop the cross-section for analysis at S335 from COI (2005). These figures highlight important changes in stratigraphy and differences in elevations at the two walls. Figure 5-2 summarizes the cross-section for S335 considered in this thesis. The results show much more refined stratigraphy (than the original one; Figure 2-7) especially the subdivision of units below the Lower Marine Clay (LMC). The design section comprises mainly low permeability clay units. However, there is a lens of higher permeability sand, F1 that intrudes across the north wall. Arup (2005) have mapped this unit very carefully, Figure 5-3, and found that it is discontinuous and therefore does not act as a source of hydraulic recharge.

The original design for M3 was based on one borehole, ABH32 and assumed a symmetric section for design of the excavation support system (Figure 2-7). The current profile is strongly asymmetric with larger changes in elevation at the base of the marine clay associated with the valley in Old Alluvium. This profile also introduces sub-units of the Kallang formation including Upper Estuarine and Lower Estuarine.
The current analysis of S335 uses the as-built data on toe embedments assumed in the M3 design sector.

Following Whittle and Davies (2006), the current analyses assume that the Upper Estuarine and Lower Estuarine clays have the same engineering properties as the UMC and LMC units, respectively. The FE model for S335 section, Figure 5-3, extends from the excavation to minimize effects of lateral boundaries on the excavation performance. 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.

5.2. Soil Layer, Diaphragm Wall & Strut Properties

Table 5-1 summarizes the MC model parameters used to present the soil and JGP layers at S335 based on values determined by Whittle and Davies (2006). It should be noted that the analyses account for linear changes in the elastic stiffness ($E'$, $\Delta E'$) and shear strength ($c'$, $\Delta c'$) of the key UMC, LMC and OA layers using a reference elevation (see Table 5-2). The Fill is treated as a drained material while other layers remain undrained through the course of the excavation.

Table 5-2a shows the elevations assumed for the upper and lower JGP layers.

---

† $K_o$ values for the UMC and LMC layers are both considered to be equal to 0.52 (instead of 0.7 as used by Whittle and Davies, 2006).
Tables 5-2b and 5-2c summarize the properties of the reinforced diaphragm wall panels† and Table 5-3 the cross-lot bracing. The later presented in terms of axial stiffness and capacity per unit width of excavation. The compression capacity of strut is based on the capacity of the strut-waler connections as reported by Bell and Chiew (2006). The first 6 levels of strutting use plate stiffnesses. These remain ductile when overloaded and hence are well modeled by elasto-plastic elements in Plaxis™. Brittle strut-waler connections were installed at levels 7-9 (using C-channel stiffness). The current analyses do not model loss of strut capacity directly, but are able to simulate collapse if the 9th level strut becomes overloaded, buy assuming full loss of capacity (as done by other experts in COI, 2005).

The initial conditions in the FE model include considerations of initial pore pressures associated with underconsolidation and deviations from K₀-conditions due to inclinations of the soil strata.

In order to represent 1-D flow condition and simulate the groundwater flow conditions observed in the field, different constant head values were assigned at different depths. Figure 5-6 shows the piezometric head profile assumed in the FE model.

The initial effective stress conditions are obtained by relaxing pre-defined K₀-conditions to bring stresses in the inclined layers into an initial drained equilibrium.

The current analyses assume \( \sigma'_p/\sigma'_{v0} = 1.0 \) in both UMC and LMC units (Fig. 5-7a). When combined with the assumed pore pressure conditions, this implies that the marine clays are

† The wall sections had three different arrangements of reinforcements and hence are each sub-divided into top, middle and lower section.
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 5-7b 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 $s_u/DSS/\sigma'_{v0} = 0.21$ 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. The figure compares the profiles obtained using the MC model (with parameter listed in Table 5-1) with results obtained when MIT-E3 model is used to represent the UMC and LMC units (and Estuarine layers). The following points should be noted:

1. The MC model generally describes higher strengths in the Upper Marine Clay than MIT-E3, but is closer to the average of the anisotropic strength in the Lower Marine Clay.

2. The MIT-E3 model predicts very similar shear strengths in the simple shear and passive (extension) modes in both UMC and LMC units.

Table 5-4 summarizes the steps used to simulate the construction sequence at S335.

† corresponding to a south section behind the south wall.
Figure 5-1a: North Wall Section Stratigraphy at M3 area (COI, 2005).
Figure 5-1b: South Wall Section Stratigraphy at M3 area (COI, 2005).
Figure 5-1: Post Collapse Site Exploration for S333, S335 & S339 Sections showing Boreholes (Arup 2005).
Figure 5-2: Extension of the F1 Fluvial Sand at the M3 Type Area (Arup, 2005).
Figure 5-3: S335 Section geometry used in FE model (Corral and Whittle, 2010).
Table 5-1: MC Model parameters for soil layers at S335 model.

<table>
<thead>
<tr>
<th>ID</th>
<th>Layer</th>
<th>Material Type</th>
<th>Reference Elevation RL, m</th>
<th>kx=ky (m/day)</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>$su$ [c'] (kPa)</th>
<th>$\Delta su/\Delta z$ (kPa/m)</th>
<th>$\phi'$ (°)</th>
<th>$\psi'$ (°)</th>
<th>$E'$ (kPa)</th>
<th>$E'/\Delta z$ (kPa/m)</th>
<th>$n'$</th>
<th>KONC</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Fill</td>
<td>drained</td>
<td>-</td>
<td>8.64E-02</td>
<td>19.0</td>
<td>[0]</td>
<td>-</td>
<td>30</td>
<td>0</td>
<td>10000</td>
<td>-</td>
<td>0.25</td>
<td>0.5</td>
</tr>
<tr>
<td>2</td>
<td>Upper MC</td>
<td>undrained</td>
<td>98.2</td>
<td>8.64E-05</td>
<td>16.0</td>
<td>20</td>
<td>0.397</td>
<td>0</td>
<td>0</td>
<td>6913</td>
<td>137</td>
<td>0.25</td>
<td>0.52</td>
</tr>
<tr>
<td>3</td>
<td>F2 Clay</td>
<td>undrained</td>
<td>-</td>
<td>8.64E-05</td>
<td>19.0</td>
<td>88</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>29250</td>
<td>-</td>
<td>0.25</td>
<td>0.7</td>
</tr>
<tr>
<td>4</td>
<td>Lower MC</td>
<td>undrained</td>
<td>83.4</td>
<td>8.64E-05</td>
<td>16.8</td>
<td>31</td>
<td>0.79</td>
<td>0</td>
<td>0</td>
<td>10310</td>
<td>346</td>
<td>0.25</td>
<td>0.52</td>
</tr>
<tr>
<td>5</td>
<td>F2-2 Clay</td>
<td>undrained</td>
<td>-</td>
<td>8.64E-05</td>
<td>20.0</td>
<td>88</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>29250</td>
<td>-</td>
<td>0.25</td>
<td>0.7</td>
</tr>
<tr>
<td>6</td>
<td>F1-Sand</td>
<td>undrained</td>
<td>-</td>
<td>8.64E-05</td>
<td>19.0</td>
<td>[0]</td>
<td>-</td>
<td>30</td>
<td>0</td>
<td>10000</td>
<td>-</td>
<td>0.25</td>
<td>0.5</td>
</tr>
<tr>
<td>7</td>
<td>OA-weathered</td>
<td>undrained</td>
<td>61.6</td>
<td>8.64E-04</td>
<td>20.0</td>
<td>100</td>
<td>52</td>
<td>0</td>
<td>0</td>
<td>33250</td>
<td>13600</td>
<td>0.25</td>
<td>1</td>
</tr>
<tr>
<td>8</td>
<td>OA-competent</td>
<td>undrained</td>
<td>-</td>
<td>8.64E-05</td>
<td>20.0</td>
<td>500</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>167500</td>
<td>-</td>
<td>0.25</td>
<td>1</td>
</tr>
<tr>
<td>9</td>
<td>JGP</td>
<td>undrained</td>
<td>-</td>
<td>8.64E-05</td>
<td>16.0</td>
<td>300</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>250000</td>
<td>-</td>
<td>0.15</td>
<td>-</td>
</tr>
</tbody>
</table>

† Linear variations with depth in stiffness and shear strength within soil layers starting at reference elevation (constant values above)
Table 5-2: (a) JGP Raft Elevations, (b) Elevations of Type Walls, and (c) Wall Material Properties (COI, 2005 and Arup, 2005).

### JGPs

<table>
<thead>
<tr>
<th>Elevations</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of Upper</td>
<td>75.3 m</td>
</tr>
<tr>
<td>Bottom of Upper</td>
<td>73.8 m</td>
</tr>
<tr>
<td>Top of Lower</td>
<td>69.2 m</td>
</tr>
<tr>
<td>Bottom of Lower</td>
<td>66.6 m</td>
</tr>
</tbody>
</table>

### Type Wall Elevations

<table>
<thead>
<tr>
<th></th>
<th>Upper</th>
<th>Middle</th>
<th>Lower</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>102.9-85.0</td>
<td>85.0-71.0</td>
<td>71.0-61.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(south)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>71.0-64.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(north)</td>
</tr>
</tbody>
</table>

### Wall Material Properties

<table>
<thead>
<tr>
<th>ID</th>
<th>Name</th>
<th>Type</th>
<th>EA [kN/m]</th>
<th>EI [kNm^2/m]</th>
<th>w [kN/m^2]</th>
<th>ν</th>
<th>Mp [kNm/m]</th>
<th>Np [kN/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>(Lower)</td>
<td>Plastic</td>
<td>160000000</td>
<td>597000</td>
<td>6.4</td>
<td>0.15</td>
<td>2300</td>
<td>1.00E+15</td>
</tr>
<tr>
<td>2</td>
<td>(Middle)</td>
<td>Plastic</td>
<td>160000000</td>
<td>597000</td>
<td>6.4</td>
<td>0.15</td>
<td>3500</td>
<td>1.00E+15</td>
</tr>
<tr>
<td>3</td>
<td>Upper</td>
<td>Plastic</td>
<td>160000000</td>
<td>597000</td>
<td>6.4</td>
<td>0.15</td>
<td>2500</td>
<td>1.00E+15</td>
</tr>
</tbody>
</table>
Table 5-3: Strut Properties & Pre-Load Struts Assumed, and Elevation (Reduced Level) for each Strut Level (Arup, 2005)

| ID | Name   | EA [kN/m] | $|F_{\text{max,comp}}|$ [kN/m] | $|F_{\text{max,tens}}|$ [kN/m] | Pre-Load [kN/m] | Elevation, RL [m] |
|----|--------|-----------|----------------|----------------|----------------|----------------|------------------|
| 1  | Level1 | 903000    | 825            | 0              | 200            | 101.9          |
| 2  | Level2 | 1148000   | 825            | 0              | 363            | 99.2           |
| 3  | Level3 | 2295000   | 1950           | 0              | 630            | 95.6           |
| 4  | Level4 | 2295000   | 1950           | 0              | 500            | 92.1           |
| 5  | Level5 | 2295000   | 1950           | 0              | 525            | 88.6           |
| 6  | Level6 | 1551000   | 1200           | 0              | 265            | 85.6           |
| 7  | Level7 | 2295000   | 1750           | 0              | 605            | 82.6           |
| 8  | Level8 | 3104000   | 2300           | 0              | 400            | 79.3           |
| 9  | Level9 | 2295000   | 1750           | 0              | 250            | 76.3           |
Figure 5-4: (a) Ground Water Heads for Initial Conditions, after Applying Initial Drained Equilibrium: (b) South Section and (c) North Section
Figure 5-5: Comparison of in situ stresses and undrained strengths of marine clay used in FE model (Corral & Whittle, 2010)
Table 5-4: Calculation Phases used in FE simulations for S335 Section.

<table>
<thead>
<tr>
<th>Phase Description</th>
<th>Elevation, RL (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Initial Drained Equilibrium (since there are no horizontal soil layers)</td>
<td></td>
</tr>
<tr>
<td>2. Diaphragm Wall Installation</td>
<td></td>
</tr>
<tr>
<td>3. JGP raft Installations</td>
<td></td>
</tr>
<tr>
<td>4. Surcharge Application of 10 kPa (outside the excavation &amp; applied at the top of the Fill)</td>
<td></td>
</tr>
<tr>
<td>5. Excavation Level 1 - Elevation</td>
<td>100.9</td>
</tr>
<tr>
<td>6. Strut Level 1 - Elevation</td>
<td>101.9</td>
</tr>
<tr>
<td>7. Excavation Level 2 - Elevation</td>
<td>98.2</td>
</tr>
<tr>
<td>8. Strut Level 2 - Elevation</td>
<td>99.2</td>
</tr>
<tr>
<td>9. Excavation Level 3 - Elevation</td>
<td>94.6</td>
</tr>
<tr>
<td>10. Strut Level 3 - Elevation</td>
<td>95.6</td>
</tr>
<tr>
<td>11. Excavation Level 4 - Elevation</td>
<td>91.1</td>
</tr>
<tr>
<td>12. Strut Level 4 - Elevation</td>
<td>92.1</td>
</tr>
<tr>
<td>13. Excavation Level 5 - Elevation</td>
<td>87.6</td>
</tr>
<tr>
<td>14. Strut Level 5 - Elevation</td>
<td>88.6</td>
</tr>
<tr>
<td>15. Excavation Level 6 - Elevation</td>
<td>84.6</td>
</tr>
<tr>
<td>16. Strut Level 6 - Elevation</td>
<td>85.6</td>
</tr>
<tr>
<td>17. Excavation Level 7 - Elevation</td>
<td>81.6</td>
</tr>
<tr>
<td>18. Strut Level 7 - Elevation</td>
<td>82.6</td>
</tr>
<tr>
<td>19. Excavation Level 8 - Elevation</td>
<td>78.3</td>
</tr>
<tr>
<td>20. Strut Level 8 - Elevation</td>
<td>79.3</td>
</tr>
<tr>
<td>21. Excavation Level 9 - Elevation</td>
<td>75.3</td>
</tr>
<tr>
<td>22. Strut Level 9 - Elevation</td>
<td>76.3</td>
</tr>
<tr>
<td>23. Excavation Level 10 - Elevation</td>
<td>72.3</td>
</tr>
<tr>
<td>24. Strut Level 10 - Elevation</td>
<td>73.3</td>
</tr>
</tbody>
</table>
6 RESULTS OF NUMERICAL SIMULATIONS FOR S335 SECTION

6.1. Computed Lateral Wall Deflections

This Chapter considers the role of the soil model on predictions of excavation performance and predictive accuracy.

Figures 6-1a and 6-1b summarize predictions of lateral wall deflections at different stages of the excavation for the South and North walls at S335. The results compare results for the MC profile with the ‘MIT-E3’ profile where UMC and LMC layers are represented using the MIT-E3 model. The main points are as follows:

1. Both sets of analyses show similar modes of wall deformation. For the South wall, there is a monotonic increase in maximum inward wall deflection as the excavation depth increases. The maximum wall deflection occurs slightly below the formation level at a given excavation stage. In contrast, the North wall deforms and reaches a maximum inward deformation when the excavation reaches El. 81.6 mRL (Level 7). For subsequent excavation there is a reversal and the wall moves back to slightly into the retained soil.

2. The MIT-E3 profile generally predicts higher inward wall movements for both South and North walls than the base case MC analyses. This result is influenced significantly by the large initial cantilever movements occurring in the first (unsupported) phase of excavation. This result reflects the very low strength at the top of the UMC unit (Figure 5-6). This condition occurs due to the assumption that OCR=1.0 throughout the marine
clay. Once struts are installed, movements of top of the wall are heavily constrained. The MIT-E3 profile shows inward movements of approximately 50 mm at the top of the North wall and 10 mm at the top of the South wall. In comparison the MC profile generates wall outward movements at the top of built North and South walls.

3. Inward movements predicted at the South wall reach a maximum movement exceeding 420 mm for MIT-E3 vs. 350 mm for the MC profile (for excavations to level 10, including the removal of the upper JGP layer). In contract, the maximum inward movement of the North wall is 195 mm and 150 mm for the MIT-E3 and MC profiles, respectively, at stage 7 (El. 81.6 mRL).

Figure 6-1 compares the computed maximum wall deflections of the two analyses. These results confirm the close qualitative agreement for the two profiles. Additionally, Table 6-1 and Figure 6-2 summarize the maximum computed lateral wall deflections with their corresponding elevations at each excavation stage.

6.2. Comparison of Computed and Measured Wall Deflection

Figure 6-3, 6-4 and 6-5 compare 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 six times during the two months preceding the collapse (with excavation depths 15.3, 18.3, 21.3, 24.6, 27.6 and 30.6 m). Figure 6-3 shows predictions for level 5 and 6 excavations (El. 87.6 and El. 84.6m), while those related to level 7 and 8 excavations (El. 81.6 and El. 78.3 mRL), and level 9 and 10 excavations are shown in Figures 6-4 and 6-5, respectively. 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 (Figure 5-6). 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 1-65 suggests that the north diaphragm wall panel remains well anchored (see Figure 6-5). 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. Table 6-2 summarizes both the computed and measured maximum wall deflections for each level excavation previously mentioned and for each side (i.e. south wall and north wall). The same information is graphically represented in Figure 6-6.

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.

6.3. Computed Bending Moments in Diaphragm Wall

Figures 6-7a and 6-7b compare computed bending moments for the MC and MIT-E3 profiles in the South and North walls, respectively. The results show relatively small differences in computed moments for the two profiles. Results for the North wall show that the design reinforcement provides adequate capacity at all stages of the excavation. The analyses of the South wall show that the plastic moment capacity is exceeded at the El. 72.3 mRL, such that a plastic hinge is predicted after removal of the upper JGP (stage 10 excavation), as indicated Figure 6-7a. This marks the outset condition for the collapse mechanism. A summary of each stage of computed maximum bending moment magnitudes is presented in Table 6-3.

6.4. Computed and Measured Strut Loads

In this section, the MC and MIT-E3 strut load predictions are discussed. All strut level elevations correspond to 1 m above the excavation levels (e.g. for excavation at EL. 100.9 mRL, strut 1 is located at EL. 101.9 mRL).

Figures 6-8, 6-10, 6-12, and Figures 6-9, 6-11, 6-13 show the strut load distribution prediction for struts 1-9 for MC and MIT-E3 profiles, respectively. A summary of the computed maximum strut load at each level is shown in Table 6-4. Two models show similar details (small
differences in magnitude). The struts were assumed to behave as an elastic-perfectly plastic material.

Figure 6-14 compares the predicted maximum strut loads for each model profile with measured data. These predicted strut loads for MC and MIT-E3 profiles are consistent with ductile buckling observed in plate stiffeners at level 6 (COI, 2005). Strut loads at level 7 and 8 are in normal ranges for MC and MIT-E3.

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.

6.5. Vertical Settlements

Figure 6-15a and 6-15b compare both the computed surface settlement profiles to the south and north of S335 using the MC and MIT-E3. The profiles generate higher settlement for MIT-E3 than MC on both sides of the excavation. Results on the north side are highly influenced by the initial cantilever deflections produced in the first stage of excavation. Also, the reverse lateral wall deflection effect on the North wall is associated with the decrease of surface North vertical settlement after level 7 excavation (El. 81.6 mRL). Table 6-5 summarizes the predicted maximum magnitudes of vertical settlements for both profiles and both wall sides.
Figure 6-1:  S335 Wall Deflection Comparison between MC and MIT-E3: (a) South Wall (b) North Wall
Table 6-1: Summary of Computed Maximum Lateral Wall Deflections & Corresponding Elevations

<table>
<thead>
<tr>
<th>Level</th>
<th>EL. RL</th>
<th>MC (m)</th>
<th>EL. (RL)</th>
<th>MIT-E3 EL.</th>
<th>MC (mm)</th>
<th>EL.</th>
<th>MIT-E3 EL.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100.9</td>
<td>25</td>
<td>102.9</td>
<td>78</td>
<td>102.9</td>
<td>159</td>
<td>102.9</td>
</tr>
<tr>
<td>2</td>
<td>98.2</td>
<td>44</td>
<td>91.9</td>
<td>87</td>
<td>93.4</td>
<td>30</td>
<td>91.9</td>
</tr>
<tr>
<td>3</td>
<td>94.6</td>
<td>71</td>
<td>90.5</td>
<td>127</td>
<td>91.1</td>
<td>47</td>
<td>89.9</td>
</tr>
<tr>
<td>4</td>
<td>91.1</td>
<td>86</td>
<td>88.4</td>
<td>148</td>
<td>88.4</td>
<td>61</td>
<td>87.2</td>
</tr>
<tr>
<td>5</td>
<td>87.6</td>
<td>105</td>
<td>86.1</td>
<td>172</td>
<td>85.6</td>
<td>83</td>
<td>83.8</td>
</tr>
<tr>
<td>6</td>
<td>84.6</td>
<td>120</td>
<td>81.9</td>
<td>192</td>
<td>82.1</td>
<td>105</td>
<td>81.9</td>
</tr>
<tr>
<td>7</td>
<td>81.6</td>
<td>183</td>
<td>79.3</td>
<td>263</td>
<td>79.6</td>
<td>126</td>
<td>79.9</td>
</tr>
<tr>
<td>8</td>
<td>78.3</td>
<td>235</td>
<td>77.3</td>
<td>311</td>
<td>77.8</td>
<td>113</td>
<td>78.3</td>
</tr>
<tr>
<td>9</td>
<td>75.3</td>
<td>268</td>
<td>75.6</td>
<td>335</td>
<td>76.3</td>
<td>98</td>
<td>76.8</td>
</tr>
<tr>
<td>10</td>
<td>72.3</td>
<td>354</td>
<td>71.3</td>
<td>447</td>
<td>71.0</td>
<td>90</td>
<td>72.8</td>
</tr>
</tbody>
</table>
Figure 6.2:
Maximum Lateral Wall Deflections vs. Elevations
Figure 6-3: Measured and Predicted Wall Deflections for Excavation Levels 5 and 6
Figure 6-4: Measured and Predicted Wall Deflections for Excavation Levels 7 and 8
Figure 6-5: Measured and Predicted Wall Deflections for Excavation Levels 9 and 10
Table 6-2: Summary of Computed and Measured Maximum Lateral Wall Deflections

<table>
<thead>
<tr>
<th>After Excavation</th>
<th>South</th>
<th>North</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EL. RL (m)</td>
<td>MC</td>
</tr>
<tr>
<td>Level</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>87.6</td>
<td>105</td>
</tr>
<tr>
<td>6</td>
<td>84.6</td>
<td>120</td>
</tr>
<tr>
<td>7</td>
<td>81.6</td>
<td>183</td>
</tr>
<tr>
<td>8</td>
<td>78.3</td>
<td>235</td>
</tr>
<tr>
<td>9</td>
<td>75.3</td>
<td>268</td>
</tr>
<tr>
<td>10</td>
<td>72.3</td>
<td>354</td>
</tr>
</tbody>
</table>
Figure 6-6: Computed and Measured Maximum Wall Deflections (a) South Wall (b) North Wall
Figure 6-7: Bending Moments for (a) South Wall, and (b) North Wall
Table 6-3: Summary of Maximum Wall Bending Moments

<table>
<thead>
<tr>
<th>Level</th>
<th>EL. RL (m)</th>
<th>After Excavation</th>
<th>South</th>
<th>North</th>
</tr>
</thead>
<tbody>
<tr>
<td>MC</td>
<td>MIT-E3</td>
<td>MC</td>
<td>MIT-E3</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>100.9</td>
<td>115</td>
<td>319</td>
<td>117</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>540</td>
</tr>
<tr>
<td>2</td>
<td>98.2</td>
<td>745</td>
<td>1108</td>
<td>624</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>984</td>
</tr>
<tr>
<td>3</td>
<td>94.6</td>
<td>1118</td>
<td>1612</td>
<td>870</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1420</td>
</tr>
<tr>
<td>4</td>
<td>91.1</td>
<td>1367</td>
<td>1747</td>
<td>1024</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1420</td>
</tr>
<tr>
<td>5</td>
<td>87.6</td>
<td>1443</td>
<td>1687</td>
<td>1030</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1274</td>
</tr>
<tr>
<td>6</td>
<td>84.6</td>
<td>1267</td>
<td>1652</td>
<td>1436</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1833</td>
</tr>
<tr>
<td>7</td>
<td>81.6</td>
<td>1996</td>
<td>2520</td>
<td>2026</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2450</td>
</tr>
<tr>
<td>8</td>
<td>78.3</td>
<td>2273</td>
<td>2736</td>
<td>1971</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2270</td>
</tr>
<tr>
<td>9</td>
<td>75.3</td>
<td>2286</td>
<td>2509</td>
<td>1431</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1632</td>
</tr>
<tr>
<td>10</td>
<td>72.3</td>
<td>2432</td>
<td>2533</td>
<td>1328</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1454</td>
</tr>
</tbody>
</table>
Figure 6-8: S335-Strut Loads for Strut Levels 1, 2 & 3, using MC
Figure 6-9: S335-Strut Loads for Strut Levels 1, 2 & 3, using MIT-E3
Figure 6-10: S335-Strut Loads for Strut Levels 4, 5 & 6, using MC
Figure 6-11: S335-Strut Loads for Strut Levels 4, 5 & 6, using MIT-E3
Figure 6-12: S335-Strut Loads for Strut Levels 7, 8 & 9, using MC
STRUT LOADS levels 7-9 - S335
MIT-E3

**Figure 6-13:** S335-Strut Loads for Strut Levels 7, 8 & 9, using MIT-E3
<table>
<thead>
<tr>
<th>#</th>
<th>Pre-Load</th>
<th>EL. RL (m)</th>
<th>MC</th>
<th>MIT-E3</th>
<th>Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>200</td>
<td>101.9</td>
<td>311</td>
<td>382</td>
<td>825</td>
</tr>
<tr>
<td>2</td>
<td>363</td>
<td>99.2</td>
<td>678</td>
<td>786</td>
<td>825</td>
</tr>
<tr>
<td>3</td>
<td>630</td>
<td>95.6</td>
<td>1093</td>
<td>1154</td>
<td>1950</td>
</tr>
<tr>
<td>4</td>
<td>500</td>
<td>92.1</td>
<td>1098</td>
<td>1099</td>
<td>1950</td>
</tr>
<tr>
<td>5</td>
<td>525</td>
<td>88.6</td>
<td>1165</td>
<td>1191</td>
<td>1950</td>
</tr>
<tr>
<td>6</td>
<td>265</td>
<td>85.6</td>
<td>1200</td>
<td>1200</td>
<td>1200</td>
</tr>
<tr>
<td>7</td>
<td>605</td>
<td>82.6</td>
<td>1492</td>
<td>1433</td>
<td>1750</td>
</tr>
<tr>
<td>8</td>
<td>400</td>
<td>79.3</td>
<td>1200</td>
<td>1162</td>
<td>2300</td>
</tr>
<tr>
<td>9</td>
<td>250</td>
<td>76.3</td>
<td>1750</td>
<td>1750</td>
<td>1750</td>
</tr>
</tbody>
</table>
Comparison of Maximum Strut Loads

Figure 6-14: Comparison of Measured and Predicted Maximum Strut Loads
Figure 6-15: Predicted Surface Vertical Settlements
Table 6-5: Predicted Maximum Surface Vertical Settlements

<table>
<thead>
<tr>
<th>Level</th>
<th>EL. RL (m)</th>
<th>South</th>
<th>North</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>MC</td>
<td>MIT-E3</td>
</tr>
<tr>
<td>1</td>
<td>100.9</td>
<td>6</td>
<td>44</td>
</tr>
<tr>
<td>2</td>
<td>98.2</td>
<td>21</td>
<td>42</td>
</tr>
<tr>
<td>3</td>
<td>94.6</td>
<td>36</td>
<td>62</td>
</tr>
<tr>
<td>4</td>
<td>91.1</td>
<td>41</td>
<td>70</td>
</tr>
<tr>
<td>5</td>
<td>87.6</td>
<td>49</td>
<td>83</td>
</tr>
<tr>
<td>6</td>
<td>84.6</td>
<td>58</td>
<td>97</td>
</tr>
<tr>
<td>7</td>
<td>81.6</td>
<td>93</td>
<td>138</td>
</tr>
<tr>
<td>8</td>
<td>78.3</td>
<td>137</td>
<td>177</td>
</tr>
<tr>
<td>9</td>
<td>75.3</td>
<td>167</td>
<td>203</td>
</tr>
<tr>
<td>10</td>
<td>72.3</td>
<td>221</td>
<td>269</td>
</tr>
</tbody>
</table>
7 CONCLUSIONS & RECOMMENDATIONS

7.1. Summary

A well-documented failure of a 30m deep braced excavation in underconsolidated marine clay using an advanced effective stress soil model (MIT-E3) is re-analyzed. 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 present analyses 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.

7.2. Conclusions

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, have been re-analyzed. 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.

7.3. Recommendations

Advanced soil model calibrations of other main units are indeed recommended. Therefore, high quality of laboratory soil tests is always required.

Accurate measurements of pore pressures must be carefully considered when a project of this magnitude is planned. This has a huge impact on predictions by numerical analyses. In addition, precise measurements of vertical settlements surrounding the excavation play a very important role.

An adequate use of monitoring data can certainly be much more useful if inverse analyses for model parameter identifications are available.
LIST OF REFERENCES


