THE DESIGN AND PERFORMANCE OF DEEP EXCAVATIONS IN CLAY

by

Lucy C. Jen

B.S. Civil Engineering, University of California, Irvine 1990
M.S. Transportation, Massachusetts Institute of Technology 1992

Submitted to the Department of Civil and Environmental Engineering
in Partial Fulfillment of the Requirements for the Degree of

DOCTOR OF PHILOSOPHY IN GEOTECHNICAL ENGINEERING

at the

Massachusetts Institute of Technology

February 1998
© 1998, Massachusetts Institute of Technology
All rights reserved

Signature of Author

[Signature]
Department of Civil and Environmental Engineering
January 15, 1998

Certified by

[Signature]
Professor Andrew J. Whittle
Associate Professor of Civil and Environmental Engineering
Thesis Co-Supervisor

Certified by

[Signature]
Professor Charles C. Ladd
Edmund K. Turner Professor of Civil and Environmental Engineering
Thesis Co-Supervisor

Accepted by

[Signature]
Professor Joseph M. Süssman
Chairman, Departmental Committee on Graduate Studies

FEB 1 3 1998
ARCHIVES
THE DESIGN AND PERFORMANCE OF DEEP EXCAVATIONS IN CLAY

by

Lucy C. Jen

Submitted to the Department of Civil and Environmental Engineering 
on January 15, 1998 in partial fulfillment of the requirements for
the Degree of Doctor of Philosophy in Geotechnical Engineering

Abstract

This thesis evaluates the performance of deep excavations in clay using coupled, 
non-linear finite element analyses, together with an effective stress soil model (MIT-E3) developed 
previously at MIT. There are three main components of the research: 1) Implementation of 
high order finite elements for simulating undrained, axisymmetric excavations in clay, and 
umerical procedures for simulating tieback bracing systems; 2) Extensive parametric studies 
to investigate how predictions of excavation-induced ground movements are related to key 
parameters including the excavation geometry, support system, and soil stress history profile. 
These calculations focus on critical sections where the supporting walls do not extend into the 
underlying bedrock; and 3) A series of three case studies in the Boston area. The predictions 
are evaluated through comparisons with measured field performance for the cross-lot braced 
excavations at South Cove and the anchored walls for the I-90 extension in South Boston The 
analyses for a third project, MBTA Transitway, were carried out as part of the initial design 
phase.

The parametric studies have established the importance of depth to bedrock as the key 
geometric parameter affecting the distribution of ground movements, while the excavation 
width, excavation depth, and uncertainties in the stress history profile and support stiffness 
are major factors contributing to the magnitudes of the displacements. The computed 
settlement troughs in the retained soil are described as dimensional functions of excavation 
deepth, wall length, bedrock depth, and soil profile. These equations offer a new approach for 
geotechnical engineers to make preliminary design calculations of ground movements.

The case studies have shown that the numerical analyses are able to achieve 
reasonably consistent predictions of wall deflections, ground movements, pore pressures, and 
strut loads for tie-back and cross-lot braced diaphragm walls. Large surface settlements 
measured at the I-90 project were attributed to tie-back installation procedures that were not 
modeled in the analyses. Much less reliable predictions of excavation performance have been 
found at sections supported by sheet pile walls. Underestimation of wall deflections appear 
to be related, in part, to longitudinal bending of the wall, which is not described by the planar 
analyses. Further three-dimensional analyses are now required to validate this hypothesis.

Thesis Co-Supervisor: Dr. Andrew J. Whittle
Title: Associate Professor of Civil and Environmental Engineering

Thesis Co-Supervisor: Dr. Charles C. Ladd
Title: Edmund K. Turner Professor of Civil and Environmental Engineering
獻給我的父母
ACKNOWLEDGMENTS

This thesis marks the end of my academic training and stay at MIT. I would like to thank my committee members for guiding and advising me during the course of my research, the teachers and professors who have made significant impressions in my life, friends who have made the entire process enjoyable, and most importantly, my family, for their unconditional support and love.

It has been a great pleasure and honor working with Professor Charles C. Ladd -- an excellent teacher, mentor, and supervisor. I will forever remember his kindness, patience, and support, especially during my last year at MIT. My appreciation to Professor Andrew J. Whittle for supervising this research and guiding me through my graduate studies. Working with Professor Whittle has been an intense and memorable learning experience both technically and personally. Sincere thanks to Professor Einstein, for being my thesis committee member. I will miss his daily "How are you?" greetings and reminders of "Get some sleep" after committee meetings. Thank you for all your great advice and most importantly, for caring.

In addition to my committee members, there are other teachers and professors who have played crucial roles in my academic training. Sincere thanks to my first English teacher, Mrs. Laurie Zanelli, at Vista Verde School in Irvine, California, for patiently teaching an 11-year old immigrant the alphabets and helping me graduate as the valedictorian of my junior high 3 years later. To Professor Roger Teal (at UC Irvine): thank you for encouraging me to pursue graduate studies; I have fond memories of your special guided tours of Newbury Street and Walden Pond! Special thanks to my MST thesis supervisor Professor Nigel Wilson for leading me to MIT and helping me become a more mature researcher. My gratitude to two people who have eased my transition to being a practicing engineer: Dr. Jack Germaine for showing me real soil (instead of computer simulations), and Mr. Fred Salvucci for playing an instrumental role in my job search process.

I am indebted to Yoshi and the entire gang at Woodward-Clyde office in Santa Ana, California, for introducing me to this fascinating field of Geotechnical Engineering. Special thanks to my boss, Mr. Henry Russell at Parsons Brinckerhoff, Boston office, for recommending South Cove as case study, and most importantly, for giving me a job! I appreciate Henry's effort of accommodating my thesis preparation schedule over the past few months; I guess you will finally see me at least 40 hours a week now.

To Alice Wong-Barlow, Shu-Ru Hsu, and Sue Ling Lai: I treasure our life-long friendship; thank you for your cheerful e-mails and letters reminding me interesting topics other than dirt and computers. To my Transportation Greek pals, Vassilis Kxpotis (Linus) and Yannis Panayotidis (Boss): the two of you have made my first two years at MIT unforgettable; I WILL visit you in Greece! To the Geotech dirtballs: Doug Cauble, Xiaomeng Yu, Shunmin Lee, Marika Santagata, Nai-Hsin Ting, Hsien-Jen Tien, Amy Varney, Mike Whelan, and Angela Alba -- thank you for all the wonderful memories! Special thanks to "Professor" Antonio (Bobi) Bobet for sharing your expertise, sense of humor, and friendship. My gratitude to Youssef Hashash, Mike Geer, and Dante Legaspi for helping me with the ABAQUS program and various versions of the user subroutines. To Professor Fushu Jeng, Dr. C. T. Chin, and Dr. Chuck Aubeny, thank you
for your professional and personal advice; I look forward working with you in this small Geotechnical Engineering world.

Special thanks to Cynthia Stewart for your patience and perfect solutions to all my academic "crisis". To Mary Elliff, thank you for feeding all the starving geotech students and organizing the best defense party for me. I will always remember your smile and your beautiful Amaryllis. To Carolyn Comer, thank you for your short lessons on art over the years, I look forward to future visits to your studio and garden.

To Cynthia Chuang, Yusin Lee, M. J. Wu, Cliff Cheng, Laura Yu, Boo Ho and Pokshim Yang, and David and Yoko Inouye, thank you for all the weekends of great food, movies, shopping, and just goofing off. Special thanks to Professor Asada in the Mechanical Engineering Department for welcoming me to all your lab's annual retreats.

I thank God for God has blessed me with the best family in the world. I dedicate this thesis to my parents, Mr. Heng-Chen Jen and Mrs. Wei-Yi Jen. I admire their extraordinary foresight and courage immigrating our entire family to the U.S. 18 years ago; the sacrifices that they've made for my siblings and myself are immeasurable. They've taught me the keys to success and happiness: honesty, moral, ethics, hard work, education, and respect. I love you very very much.

I thank my sister, Joanna, for setting an extremely high standard and great example for me to follow. I thank my brother, Francis, for always being there for me and making sure that I have everything that I need and want. Deepest gratitude to my brother Stephen, for leading me to MIT and always encouraging me to set and attain high goals.

Finally, to my husband, Sheng Liu, thank you for your love, strength, sensitivity, and sense of humor. Without your emotional support (and great dinners), completion of this dissertation would not be possible. I cannot imagine my life without you. I love you with all my heart and I look forward to spending the rest of our lives together.
TABLE OF CONTENTS

Abstract 3
Acknowledgments 5
Table of Contents 7
List of Tables 11
List of Figures 13

I. INTRODUCTION 33
   1.1 Expansion of Numerical Techniques 34
   1.2 Parametric Study 35
   1.3 Case Studies 36
   1.4 Development of Design Recommendations 37

II. DEVELOPMENT OF THE FINITE ELEMENT CAPABILITIES FOR ANALYZING EXCAVATIONS IN CLAY 39
   2.1 Factors Influencing Excavation Performance in Clay 41
   2.2 Summary of Current Design Practices for Excavations in Clay 42
      2.2.1 Prediction of Soil Movements 43
         2.2.1.1 Empirical Approach 44
         2.2.1.2 Numerical Analysis 48
      2.2.2 Design of Supporting Structures 49
         2.2.2.1 Excavation Support Wall 50
         2.2.2.2 Excavation Wall Support 51
   2.3 Previous Finite Element Analyses Performed at MIT 53
      2.3.1 Description of the MIT-E3 Model 54
      2.3.2 Implementation of the MIT-E3 model in ABAQUS 55
   2.4 Description of Current Work 55
      2.4.1 Modifications of the Excavation Procedure in ABAQUS 57
      2.4.2 Implementation of the User Elements for Axisymmetric Undrained Analyses 61
         2.4.2.1 ABAQUS UEL Subroutine for Undrained Axisymmetric Analysis 61
         2.4.2.2 Modifications to the Original UEL Subroutine 62
         2.4.2.3 Comparison of Axisymmetric and Plane Strain Excavations 65
   2.4.3 Modeling of Support Systems 66
      2.4.3.1 Support Wall Stiffness 67
      2.4.3.2 Modeling of Passive and Pre-stressed Struts 68
      2.4.3.3 Modeling of Tiebacks 69
2.5 Application of Finite Element Capabilities

III. PARAMETRIC ANALYSES OF FACTORS CONTROLLING EXCAVATION-INDUCED GROUND MOVEMENTS

3.1 Summary of Previous Parametric Analyses
   3.1.1 Model Assumption
   3.1.2 Summary of Results

3.2 Excavation Geometry
   3.2.1 Scope of Experiments for Excavation Geometry
   3.2.2 Group A: Effect of Wall Length
   3.2.3 Group B: Effect of Excavation Width
   3.2.4 Group C: Effect of Depth to Bedrock
   3.2.5 Summary of Excavation Geometry Effects

3.3 Soil Profile
   3.3.1 Group D: Variations in Clay Strength/OCR Profile
   3.3.2 Group E: Effects of Overlying Cohesionless Material
   3.3.3 Group F: Effects of Clay Crust
   3.3.4 Summary of Soil Profile Effects

3.4 Structural Support System for Excavations
   3.4.1 Group G: Effects of Wall Stiffness
      3.4.1.1 Results of Group G1 Experiments
      3.4.1.2 Results of Group G2 Experiments
   3.4.2 Group H: Effects of Strut Stiffness
      3.4.2.1 Results for Group H1 Experiments
      3.4.2.2 Results for Group H2 Experiments
      3.4.2.3 Results for Group H3 Experiments
   3.4.3 Summary of Effects of Wall Stiffness and Strut Stiffness

IV. ANALYSES OF THREE DEEP EXCAVATIONS IN BOSTON

4.1 South Cove Case Study
   4.1.1 South Cove Site Description
   4.1.2 Soil Profile
   4.1.3 Field Instruments
   4.1.4 Construction Sequence
      4.1.4.1 Section A-A
      4.1.4.2 Section B-B
   4.1.5 Finite Element Model and Input Parameters
      4.1.5.1 In-Situ Conditions
      4.1.5.2 Finite Element Model
   4.1.6 Numerical Results for Section A-A
   4.1.7 Numerical Results for Section B-B
   4.1.8 Summary of Findings from the South Cove Study

4.2 Deep Excavation in South Boston
   4.2.1 Site Description
   4.2.2 Subsurface Conditions
   4.2.3 Field Instruments
4.2.4 Construction Activities 395
4.2.5 Finite Element Model 396
4.2.6 Base Case Analysis 400
   4.2.6.1 Wall Deflections and Surface Settlements at 401
   each Construction Step
   4.2.6.2 Time History Comparisons 406
4.2.7 Assessment of Base Case Analysis 411
   4.2.7.1 Approximations in Finite Element Model 413
   4.2.7.2 Material Properties 418
4.2.8 Revised Case Analysis 421
   4.2.8.1 Construction Step Comparisons 421
   4.2.8.2 Time History Comparisons 423
4.2.9 Evaluation of the South Boston Excavation Case Study 424

4.3 The Transitway Project 427
4.3.1 Site Description 427
4.3.2 Soil Profile 429
   4.3.2.1 Soil Properties for the Upper Soil Strata 429
   4.3.2.2 Soil Properties for the Boston Blue Clay Stratum 430
4.3.3 Proposed Construction Sequence 431
4.3.4 Finite Element Model and Input Parameters 433
   4.3.4.1 Finite Element Mesh 433
   4.3.4.2 Element Type 434
   4.3.4.3 Material Models and Input Properties 434
   4.3.4.4 Summary of Numerical Analyses 436
4.3.5 Numerical Results for Platform Level 437
   4.3.5.1 Effect of Strut Spacing and Wall Length 437
   4.3.5.2 Effect of Partial Drainage 440
   4.3.5.3 Effect of Clay Strength Properties 442
4.3.6 Numerical Results for the Mezzanine Level 444
4.3.7 Numerical Results for the Transition Section 446
4.3.8 Numerical Results for the West Tunnel 448
4.3.9 Evaluation of the Proposed Excavation for the 449
   Transitway Project

4.4 Evaluation of the Predictive Capabilities and Limitations 451
   of the Numerical Analyses

V. PREDICTION OF SURFACE SETTLEMENTS FOR EXCAVATIONS 631
   IN CLAY-DOMINATED PROFILES

5.1 Predictions of Settlement for Excavations in Normally 632
   Consolidated BBC with various Excavation Geometries 632
   5.1.1 Normalized Surface Settlement 634
   5.1.2 Maximum Surface Settlement 636
   5.1.3 Prediction of Surface Settlements for Excavations in 638
   Normally Consolidated BBC

5.2 Predictions of Settlement Various Soil Profiles 638
   5.2.1 Distribution of Surface Settlement in Composite Profile 639
5.2.2 Magnitude of Maximum Surface Settlement in a Composite Profile 639
5.2.3 Predicted Surface Settlement for Excavations in a Composite Profile 642
5.3 Predictions of Settlement for Various Support Systems 642
  5.3.1 Effect of Support System on Distribution of Surface Settlements 643
  5.3.2 Maximum Surface Settlement for Various Support System Stiffness 643
  5.3.3 Surface Settlement for Various Support System Stiffness 646
5.4 Summary of Design Recommendations for Predicting Surface Settlements 646

VI. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS 675
  6.1 Development of FEA Capabilities of Analyzing Excavations in Clay 676
  6.2 Parametric Study 678
  6.3 Case Studies 680
  6.4 Prediction of Surface Settlement 683
  6.5 Recommendations for Future Work 683

List of References 687

APPENDIX A - Sample ABAQUS Input Files and User Subroutines 693
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1.1</td>
<td>Typical Factors Cited which Influence Excavation Performance (modified based on Mana and Clough, 1981)</td>
<td>73</td>
</tr>
<tr>
<td>2.2.1</td>
<td>Summary of Methods Used to Estimate Prestress Load on Tieback Anchors [from Xanthakos, 1991]</td>
<td>73</td>
</tr>
<tr>
<td>2.4.1</td>
<td>Comparison of Computation Time for a Model Excavation using Three Different Workstations</td>
<td>74</td>
</tr>
<tr>
<td>2.4.2</td>
<td>Typical Properties and Dimensions of Tieback in Clay</td>
<td>74</td>
</tr>
<tr>
<td>3.1.2</td>
<td>Summary of Factors Analyzed in the Parametric Study</td>
<td>166</td>
</tr>
<tr>
<td>3.1.3</td>
<td>Summary of Parametric Study Performed by Hashash and Whittle [1996]</td>
<td>166</td>
</tr>
<tr>
<td>3.1.4</td>
<td>Coefficients for Interpolating Maximum Wall and Ground Movements [Hashash and Whittle, 1996]</td>
<td>167</td>
</tr>
<tr>
<td>3.2.1</td>
<td>Summary of Principal Parameters in Hashash's Parametric Study and the new Numerical Experiments on Wall Length, Excavation Width, and Depth to Bedrock covered in Sections 3.2.2, 3.2.3, and 3.2.4</td>
<td>168</td>
</tr>
<tr>
<td>3.2.2</td>
<td>Summary of Cases Evaluated in Current Parametric Analyses of Excavation Geometry [OCR = 1.0, h = 2.5-m, 0.9-m concrete diaphragm wall]</td>
<td>169</td>
</tr>
<tr>
<td>3.2.3</td>
<td>Summary of Results of Group C Experiments Presented in Section 3.2.4 -- Evaluation of Depth to Bedrock Effects</td>
<td>170</td>
</tr>
<tr>
<td>3.2.4</td>
<td>Summary of Results for the Excavation Geometry Finite Element Experiments</td>
<td>171</td>
</tr>
<tr>
<td>3.3.1</td>
<td>Typical Engineering Properties of Foundation Material in Boston (from Johnson, 1989)</td>
<td>172</td>
</tr>
<tr>
<td>3.3.2</td>
<td>Summary of Soil Profiles and Excavation Geometries at Five Excavation Project Sites in Boston, MA</td>
<td>173</td>
</tr>
<tr>
<td>3.3.3</td>
<td>Summary of Results for the Soil Profile Finite Element Experiments</td>
<td>174</td>
</tr>
<tr>
<td>3.4.1</td>
<td>Summary of Support Wall Dimensions and Properties [G1]</td>
<td>175</td>
</tr>
<tr>
<td>3.4.2</td>
<td>Summary of Results for Groups G1 and G2 Analyses</td>
<td>175</td>
</tr>
<tr>
<td>3.4.3</td>
<td>Summary of Failure Depths for Group G1 Analyses</td>
<td>176</td>
</tr>
<tr>
<td>3.4.4</td>
<td>Summary of Final Excavation Depth due to Soil Failure in Group G2 Analyses</td>
<td>176</td>
</tr>
<tr>
<td>3.4.5</td>
<td>Summary of Final Excavation Depth due to Structural Failure of the Wall for Group G2 Analyses</td>
<td>176</td>
</tr>
<tr>
<td>3.4.6</td>
<td>Summary of Strut Stiffness Considered in Group H1 Analyses</td>
<td>177</td>
</tr>
<tr>
<td>Table</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>-------</td>
<td>-----------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>Table 3.4.7</td>
<td>Summary of Results for Groups H1, H2, and H3 Analyses</td>
<td>177</td>
</tr>
<tr>
<td>Table 3.4.8</td>
<td>Summary of Structural Forces for Group H1 Analyses</td>
<td>178</td>
</tr>
<tr>
<td>Table 3.4.9</td>
<td>Summary of Structural Forces for Group H2 Analyses</td>
<td>178</td>
</tr>
<tr>
<td>Table 3.4.10</td>
<td>Summary of Structural Forces for Group H3 Analyses</td>
<td>179</td>
</tr>
<tr>
<td>Table 3.4.11</td>
<td>Summary of Results for the Support System Finite Element Analyses</td>
<td>180</td>
</tr>
<tr>
<td>Table 4.1.1</td>
<td>Elastic Properties of Cross-Lot Bracing at South Cove</td>
<td>455</td>
</tr>
<tr>
<td>Table 4.2.1</td>
<td>Summary of Tieback Dimensions at ISS-4 [Whelan, p. 141, 1995]</td>
<td>456</td>
</tr>
<tr>
<td>Table 4.2.2</td>
<td>Summary of Effective Young's Moduli and Lock-off Loads of Tiebacks at ISS-4 [Whelan, p. 141, 1995]</td>
<td>456</td>
</tr>
<tr>
<td>Table 4.2.3a</td>
<td>Recommended Soil Engineering Properties for ISS-4 [after Whelan, 1995, Tables 5.1 and 5.2]</td>
<td>457</td>
</tr>
<tr>
<td>Table 4.2.3b</td>
<td>Recommended Soil Engineering Properties for ISS-4 [after Whelan, 1995, Tables 5.1 and 5.2]</td>
<td>458</td>
</tr>
<tr>
<td>Table 4.2.4</td>
<td>Geotechnical Field Instruments at ISS-4, Boston, MA</td>
<td>459</td>
</tr>
<tr>
<td>Table 4.2.5</td>
<td>Soil Model and Input Material Properties for Soils in the Upper Aquifer</td>
<td>460</td>
</tr>
<tr>
<td>Table 4.2.6</td>
<td>Input Stress History and In-Situ Stress State for BBC layer using MIT-E3 Soil Model</td>
<td>460</td>
</tr>
<tr>
<td>Table 4.2.7</td>
<td>Properties and Input Parameters for Structural Elements of Lateral Earth Support System</td>
<td>461</td>
</tr>
<tr>
<td>Table 4.2.8</td>
<td>Summary of Actual Construction Sequence and Numerical Model Construction Sequence Definitions</td>
<td>462</td>
</tr>
<tr>
<td>Table 4.2.9</td>
<td>Listing of Figures Showing Results of the Base and Revised Case Numerical Analyses</td>
<td>463</td>
</tr>
<tr>
<td>Table 4.3.1</td>
<td>Summary of Excavation Geometries for the Four Typical Cross-Sections at the Transitway Project</td>
<td>464</td>
</tr>
<tr>
<td>Table 4.3.2</td>
<td>Properties and Model Parameters of Upper Soil Layers</td>
<td>465</td>
</tr>
<tr>
<td>Table 4.3.3</td>
<td>Properties and Input Parameters for Structural Elements of Lateral Earth Support System</td>
<td>466</td>
</tr>
<tr>
<td>Table 4.3.4</td>
<td>Summary of Finite Element Analyses Performed for the MBTA Transitway Excavations</td>
<td>467</td>
</tr>
<tr>
<td>Table 4.3.5</td>
<td>Summary of Maximum Displacements for the Transitway Project</td>
<td>467</td>
</tr>
<tr>
<td>Table 4.3.6</td>
<td>Summary of Structural Loads for the Transitway Project</td>
<td>468</td>
</tr>
</tbody>
</table>
LIST OF FIGURES

Figure 2.1.1 Factors Influencing Ground Movements due to Excavations in Soft Ground 75
Figure 2.2.1 Summary of Observed Settlements behind Sheet Pile and Soldier Pile Walls [Peck, 1969] 76
Figure 2.2.2 Relationship between Factor of Safety Against Basal Heave and Normalized Maximum Lateral Wall Movements from Case History Data [Mana and Clough, 1981] 77
Figure 2.2.3 Relationship between Maximum Ground Settlement and Maximum Lateral Wall Movement based on Case History Data [Mana and Clough, 1981] 77
Figure 2.2.4 Recommended Adjustment Factors for Effects of Strut Stiffness, Effect of Depth to Underlying Firm Layer, and Effect of Excavation Width [Mana and Clough, 1981] 78
Figure 2.2.5 Relation Among Maximum Lateral Wall Movements, System Stiffness, and Factor of Safety Against Basal Heave for Cuts in Plastic Clay [from Terzaghi et al., 1996, p. 462]: (a) Calculated by Finite-Element Solutions; (b) Comparison with Field Measurements [after Clough et al., 1989] 79
Figure 2.2.6 Envelopes for Normalized Ground Settlement Profiles [Mana and Clough, 1981] 80
Figure 2.2.7 Dimensionless Settlement Profiles Recommended for Estimating Settlement Distribution Adjacent to Excavations in Different Soil Types [Clough and O'Rourke, 1990] 81
Figure 2.2.8 Active Force and Passive Resistance in Wall assuming Free-Earth Support Conditions [Xanthakos, 1994] 82
Figure 2.2.9 Apparent Earth Pressure Diagrams for Computing Strut Loads in Strutter Excavations [Peck, 1969] 82
Figure 2.2.10 Modes of Failure for Tieback Anchored Walls and Analysis Methods for Evaluating the Stability of the Wall 83
Figure 2.3.1a Yield, Failure, and Load Surfaces used in MIT-E3 [Whittle and Kavvadas, 1994] 84
Figure 2.3.1b Effects of Stress History on Initial Values of State Variables in MIT-E3 Model [Whittle and Kavvadas, 1994] 84
Figure 2.3.2 Evaluation of Stress Paths Predicted by MIT-E3 and MCC Soil Models for Undrained Plane Strain Shear 84
of K0-Normally Consolidated BBC in Directional Shear Cell [Whittle et al, 1994]

Figure 2.3.3 Evaluation of Shear Stress-Strain and Secant Shear Modulus-Strain Response of K0-Normally Consolidated BBC in Directional Shear Cell Tests Using MIT-E3 and MCC Soil Models [Whittle et al., 1994] 85

Figure 2.4.1 2-D Mixed Elements used in Finite Element Analysis of Excavations 86

Figure 2.4.2 Wall Deflections and Surface Settlements for Model Plane Strain and Axisymmetric Undrained Excavations at H = 12.5-m 87

Figure 2.4.3 Derivation of Equivalent Plane Strain Strut Stiffness 88

Figure 2.4.4 Modeling of Passive Strut within Finite Element Analysis 89

Figure 2.4.5 Modeling of Pre-stressed Strut within FE Analysis 90

Figure 2.4.6 Numerical Model of Tieback anchored in Bedrock 91

Figure 2.4.7 Numerical Modeling of Tiebacks and Application of Lock-off Load for Tieback Anchors in Clay 92

Figure 2.4.8 Element Definition for Modeling the Free Length of the Tieback Anchors in Clay using ABAQUS 93

Figure 2.4.9 Application of Tieback Lock-off Load for Tieback Anchors in Clay using ABAQUS 94

Figure 2.4.10 Calculation of Equivalent Stiffness for Tieback Anchors in Clay 95

Figure 2.4.11 Numerical Experiments Illustrating the Effects of Lock-off Load 96

Figure 2.4.12 Wall Deflections and Surface Settlements of Excavations with Varying Levels of Lock-off Load at H = 5-m 97

Figure 2.4.13 Wall Deflections and Surface Settlements of Excavations with Varying Levels of Lock-off Load at H = 7.5m 98

Figure 2.4.14 Lateral Movements at Tieback Anchorage and Head of the Fixed Length as Functions of Lock-off Force and Excavation Depth 99

Figure 2.4.15 Vertical Movements at Tieback Anchorage and Head of the Fixed Length as Functions of Lock-off Force and Excavation Depth 100

Figure 3.1.1 Finite Element Mesh and Boundary Conditions for Hashash’s Numerical Study [Hashash, 1992, p. 158] 101

Figure 3.1.2 Initial Conditions and Finite Element Model Excavation Sequence for Hashash’s Numerical Study [Hashash, 1992, p.157] 181

Figure 3.1.3 Effect of Wall Length on Maximum Wall Bending Moments, Deflections, and Ground Movements [from Hashash and Whittle, 1996] 182

Figure 3.1.4 Effect of Wall Length on Lateral Deflections and 183
Surface Settlements for OCR = 1.0 Clay Profile [from Hashash and Whittle, 1996]

Figure 3.1.5 Effect of Support Spacing on Lateral Wall Deflections and Surface Settlements for OCR = 1.0 Clay Profile [from Hashash and Whittle, 1996]

Figure 3.1.6 Effect of Support Spacing on Maximum Lateral Wall Deflections and Bending Moments [from Hashash and Whittle, 1996]

Figure 3.1.7 Comparison of Composite Soil Profile with Measured Data in South Boston [from Hashash and Whittle, 1996]

Figure 3.1.8 Summary of Excavation Behavior for Overconsolidated Clay Profiles [from Hashash and Whittle, 1996]

Figure 3.1.9 Comparison of Maximum Wall Deflections and Bending Moments for Composite and Constant OCR Profiles [from Hashash and Whittle, 1996]

Figure 3.1.10 Estimation of Maximum Lateral Wall Deflections from Numerical Experiments [from Hashash and Whittle, 1996]

Figure 3.2.1 Scope of Parametric Study for Excavations in Clay Presented in Sections 3.1 and 3.2

Figure 3.2.2 Measures of Ground Movements and Forces on Structural Support System

Figure 3.2.3 Deflected Wall Shape from Group A Experiments

Figure 3.2.4 Surface Settlements from Group A Experiments

Figure 3.2.5 Surface Horizontal Displacements from Group A Experiments

Figure 3.2.6 Heave within the Excavation from Group A Experiments

Figure 3.2.7a Strut Loads from Group A Experiments (Struts 1-4)

Figure 3.2.7b Strut Loads from Group A Experiments (Struts 5-7)

Figure 3.2.8 Moment Distribution in the Wall from Group A Experiments: H = 2.5-m to 12.5-m

Figure 3.2.9 Moment Distribution in the Wall from Group A Experiments: H = 12.5-m to 17.5-m

Figure 3.2.10 Total Horizontal Stress, Effective Horizontal Stress, and Pore Pressure for Group A Experiments at H = 5.0-m

Figure 3.2.11a Total Horizontal Stress for Group A Experiments at H = 15-m

Figure 3.2.11b Effective Horizontal Stress for Group A Experiments at H = 15-m

Figure 3.2.11c Pore Pressure for Group A Experiments at H = 15-m

Figure 3.2.12 Group B Experiments for the Evaluation of the Excavation Width (B) Effects

Figure 3.2.13 Deflected Wall Shape from Group B Experiments at H = 17.5-m
Figure 3.2.14  Maximum Wall Deflections as Functions of the Excavation Widths from Group B Experiments

Figure 3.2.15  Surface Settlements from Group B Experiments at H = 17.5-m

Figure 3.2.16  Maximum Surface Settlements as Functions of Excavation Widths from Group B Experiments

Figure 3.2.17  Surface Horizontal Displacements from Group B Experiments at H = 17.5-m

Figure 3.2.18  Maximum Surface Horizontal displacements as Functions of Excavation Widths for Group B Experiments

Figure 3.2.19  Heave within the Excavation from Group B Experiments at H = 17.5-m

Figure 3.2.20  Maximum Heave within Excavation as Functions of Excavation Widths from Group B Experiments

Figure 3.2.21  Equivalent Horizontal Pressure Calculated based on Strut Loads from Group B Experiments at H = 17.5-m

Figure 3.2.22  Moment Distribution in the Wall from Group B Experiments at H = 7.5-m and 17.5-m

Figure 3.2.23  Total Horizontal Stress, Effective Horizontal Stress, and Pore Pressure for Group B Experiments at H = 17.5-m

Figure 3.2.24  Group C Numerical Experiments for the Evaluation of the Bedrock Depth (d_B) Effects

Figure 3.2.25  Wall Deflections and Surface Settlements for Group C1 Experiments

Figure 3.2.26a  Wall Deflections and Surface Settlements for Group C2 Experiments (d_B = 30-m, 35-m, and 50-m)

Figure 3.2.26b  Wall Deflections and Surface Settlements for Group C2 Experiments (d_B = 50-m, 75-m, and 100-m)

Figure 3.2.27  Wall Deflections and Surface Settlements for Group C3 Experiments

Figure 3.2.28a  Wall Deflections and Surface Settlements for Group C4 Experiments (d_B = 30-m, 35-m, and 50-m)

Figure 3.2.28b  Wall Deflections and Surface Settlements for Group C4 Experiments (d_B = 50-m, 75-m, and 100-m)

Figure 3.2.29  Wall Deflections and Surface Settlements for Group C5 Experiments

Figure 3.2.30a  Wall Deflections and Surface Settlements for Group C6 Experiments (d_B = 30-m, 35-m, and 50-m)

Figure 3.2.30b  Wall Deflections and Surface Settlements for Group C6 Experiments (d_B = 50-m, 75-m, and 100-m)

Figure 3.2.31a  Normalized Settlement Troughs for Group C Experiments at H = 7.5-m

Figure 3.2.31b  Normalized Settlement Troughs for Group C Experiments at H = 15-m

Figure 3.2.32a  Normalized Maximum Settlements and Maximum Wall Deflections from Group C Experiments (H = 7.5-m)
| Figure 3.2.32b | Normalized Maximum Settlements and Maximum Wall Deflections from Group C Experiments (H = 17.5-m) | 235 |
| Figure 3.2.33 | Surface Horizontal Displacements for Group C1 Experiments | 236 |
| Figure 3.2.34 | Surface Horizontal Displacements for Group C2 Experiments | 237 |
| Figure 3.2.35 | Surface Horizontal Displacements for Group C3 Experiments | 238 |
| Figure 3.2.36 | Surface Horizontal Displacements for Group C4 Experiments | 239 |
| Figure 3.2.37 | Surface Horizontal Displacements for Group C5 Experiments | 240 |
| Figure 3.2.38 | Surface Horizontal Displacements for Group C6 Experiments | 241 |
| Figure 3.2.39 | Heave within Excavation for Group C1 Experiments | 242 |
| Figure 3.2.40 | Heave within Excavation for Group C2 Experiments | 243 |
| Figure 3.2.41 | Heave within Excavation for Group C3 Experiments | 244 |
| Figure 3.2.42 | Heave within Excavation for Group C4 Experiments | 245 |
| Figure 3.2.43 | Heave within Excavation for Group C5 Experiments | 246 |
| Figure 3.2.44 | Heave within Excavation for Group C6 Experiments | 247 |
| Figure 3.2.45a | Strut Loads from Group C1 Experiments (Struts 1 - 4) | 248 |
| Figure 3.2.45b | Strut Loads from Group C1 Experiments (Struts 5 - 7) | 249 |
| Figure 3.2.46a | Strut Loads from Group C2 Experiments (Struts 1 - 4) | 250 |
| Figure 3.2.46b | Strut Loads from Group C2 Experiments (Struts 5 - 7) | 251 |
| Figure 3.2.47a | Strut Loads from Group C3 Experiments (Struts 1 - 4) | 252 |
| Figure 3.2.47b | Strut Loads from Group C3 Experiments (Struts 5 - 7) | 253 |
| Figure 3.2.48 | Moment Distributions in the Wall from Group C1 Experiments | 254 |
| Figure 3.2.49 | Moment Distributions in the Wall from Group C3 Experiments | 255 |
| Figure 3.2.50 | Total Horizontal Stress, Effective Horizontal Stress, and Pore Pressure for Group C1 at H = 15-m | 256 |
| Figure 3.2.51 | Total Horizontal Stress, Effective Horizontal Stress, and Pore Pressure for Group C3 at H = 15-m | 257 |
| Figure 3.3.1 | Summary of Numerical Analyses used to Evaluate the Impacts of three types of Variations in Soil Profile | 258 |
| Figure 3.3.2 | Specification of Group D Experiments for Evaluating the Effects of Clay Stress History | 259 |
| Figure 3.3.3 | Wall Deflections for Group D Experiments with OCR = 1.15 Clay | 260 |
| Figure 3.3.4 | Surface Settlements for Group D Experiments with OCR = 1.15 Clay | 261 |
| Figure 3.3.5 | Wall Deflections for Group D Experiments with OCR = 1.25 Clay | 262 |
| Figure 3.3.6 | Surface Settlements for Group D Experiments with OCR = 1.25 Clay | 263 |
Figure 3.3.7  Wall Deflections for Group D Experiments with OCR = 1.7 Clay
Figure 3.3.8  Surface Settlements for Group D Experiments with OCR = 1.7 Clay
Figure 3.3.9  Wall Deflections for Group D Experiments with OCR = 2.0 Clay
Figure 3.3.10 Surface Settlements for Group D Experiments with OCR = 2.0 Clay
Figure 3.3.11 Normalized Maximum Wall Deflections and Surface Settlements for Group D Experiments
Figure 3.3.12 Normalized Deflected Wall Shapes for Group D Experiments at H = 2.5, 5.0, 12.5, and 17.5-m
Figure 3.3.13a Normalized Surface Settlements for Group D Experiments at H = 2.5-m and 12.5-m
Figure 3.3.13b Normalized Surface Settlements for Group D Experiments at H = 5.0-m and 17.5-m
Figure 3.3.14 Moment Distributions in the Wall for Group D Experiments at H = 17.5-m
Figure 3.3.15 Equivalent Horizontal Stress Calculated from Strut Loads for Group D Experiments at H = 17.5-m
Figure 3.3.16a Typical Soil Profile in the Downtown Boston Area [Johnson, 1989]
Figure 3.3.16b Idealized Soil Profiles for the Numerical Experiments in Group E
Figure 3.3.17 Group E Numerical Experiments for the Evaluation of the Overlying Cohesionless Soil Effects
Figure 3.3.18 Wall Deflections from the Group E Experiments with Cohesionless Soil Overlying OCR = 1 Clay
Figure 3.3.19 Surface Settlements for Group E Experiments with Cohesionless Soil Overlying OCR 1 Clay
Figure 3.3.20 Wall Deflections for Group E Experiments with Cohesionless Soil Overlying OCR = 1.15 Clay
Figure 3.3.21 Surface Settlements for Group E Experiments with Cohesionless Soil Overlying OCR 1.15 Clay
Figure 3.3.22 Contours of Vertical Displacements, $u_y$, for an Excavation with $B = 40$-m, $L = 25$-m, $d_B = 50$-m, $h = 2.5$-m, and OCR = 1.0 at Excavation Depths of 5.0-m, 12.5-m, 15-m, and 17.5-m
Figure 3.3.23 Contours of Vertical Displacements, $u_y$, for an Excavation with $B = 40$-m, $L = 25$-m, $d_B = 50$-m, $h = 2.5$-m, and Profile E1 at Excavation Depths of 5.0-m, 12.5-m, 15-m, and 17.5-m
Figure 3.3.24 Contours of Vertical Displacements, $u_y$, for an Excavation with $B = 40$-m, $L = 25$-m, $d_B = 50$-m, $h = 2.5$-m, and Profile E2 at Excavation Depths of 5.0-m, 12.5-m, 15-m, and 17.5-m
Figure 3.3.25  Contours of Vertical Displacements, $u_y$, for an Excavation with $B = 40$-m, $L = 25$-m, $d_B = 50$-m, $h = 2.5$-m, and OCR = 1.15 at Excavation Depths of 5.0-m, 12.5-m, 15-m, and 17.5-m

Figure 3.3.26  Contours of Vertical Displacements, $u_y$, for an Excavation with $B = 40$-m, $L = 25$-m, $d_B = 50$-m, $h = 2.5$-m, and Profile E3 at Excavation Depths of 5.0-m, 12.5-m, 15-m, and 17.5-m

Figure 3.3.27  Contours of Vertical Displacements, $u_y$, for an Excavation with $B = 40$-m, $L = 25$-m, $d_B = 50$-m, $h = 2.5$-m, and Profile E4 at Excavation Depths of 5.0-m, 12.5-m, 15-m, and 17.5-m

Figure 3.3.28  Moment Distributions in the Wall for Group E Experiments with Underlying OCR = 1.0 Clay (Profiles E1 and E2)

Figure 3.3.29  Moment Distributions in the Wall for Group E Experiments with Underlying OCR = 1.15 Clay (Profiles E3 and E4)

Figure 3.3.30  Strut Loads for Group E Experiments with Underlying OCR = 1.0 Clay (Profiles E1 and E2)

Figure 3.3.31  Strut Loads for Group E Experiments with Underlying OCR = 1.15 Clay (Profiles E3 and E4)

Figure 3.3.32  Stress History Profile Measured at the South Boston Special Test Site using 1-D Consolidation Tests

Figure 3.3.33  Undrained Shear Strength Profile Measured at the South Boston Special Test Site

Figure 3.3.34  Stress History Profiles (F1, F2, F3, and F4) included in Group F Numerical Analysis: Clay Crust Effects

Figure 3.3.35  Group F Numerical Experiments for the Evaluation of the Clay Crust Effects

Figure 3.3.36  Wall Deflections for Group F Experiments with Soil Profiles F1 and F2

Figure 3.3.37  Subsurface Settlements from Group F Experiments with Soil Profiles F1 and F2

Figure 3.3.38  Wall Deflections from Group F Experiments with Soil Profiles F1 and F3

Figure 3.3.39  Surface Settlements from Group F Experiments with Soil Profiles F1 and F3

Figure 3.3.40  Wall Deflections for Group F Experiments with Soil Profiles F3 and F4

Figure 3.3.41  Surface Settlements from Group F Experiments with Soil Profiles F3 and F4

Figure 3.3.42  Moment Distribution in the Wall for Group F Experiments with Soil Profiles F1, F2, F3, and F4

Figure 3.3.43  Strut Loads for Group F Experiments with Soil Profiles F1, F2, F3, and F4
Figure 3.4.1  Group G Numerical Experiments for the Evaluation of the Support Structure 302
Figure 3.4.2  Group G1 Numerical Experiments for the Evaluation of the Effects of Support Wall Stiffness 303
Figure 3.4.3  Wall Deflections and Surface Settlements for Group G1 Experiments at H = 5.0-m 304
Figure 3.4.4  Wall Deflections and Surface Settlements for Group G1 Experiments at H = 7.5 305
Figure 3.4.5  Wall Deflections and Surface Settlements for Group G1 Experiments at H = 10.0-m 306
Figure 3.4.6  Wall Deflections and Surface Settlements for Group G1 Experiments at H = 12.5 307
Figure 3.4.7  Summary of Maximum Displacements as Function of Excavation Depth for Group G1 Analyses 308
Figure 3.4.8  Summary of Maximum Displacements as Function of the Bending Stiffness of the Support Wall for Group G1 Analyses 309
Figure 3.4.9  Moment Distribution in the Wall for Group G1 Experiments at H = 10-m 310
Figure 3.4.10  Summary of Strut Forces for Group G1 Analyses, OCR = 1.0, L = 25-m, B = 40-m, d_B = 50-m, and Perfect Struts at h = 2.5-m 311
Figure 3.4.11  Group G2 Numerical Experiments for the Evaluation of the Combined Effects of Support Wall Stiffness and Soil Profile 313
Figure 3.4.12  Wall Deflections and Surface Settlements for Group G2 Experiments with Soil Profile F1 314
Figure 3.4.13  Wall Deflections and Surface Settlements for Group G2 Experiments with Soil Profile F2 315
Figure 3.4.14  Wall Deflections and Surface Settlements for Group G2 Experiments with Soil Profile F3 316
Figure 3.4.15  Wall Deflections and Surface Settlements for Group G2 Experiments with Soil Profile F4 317
Figure 3.4.16  Summary of Maximum Displacements as Functions of Excavation Depth for Group G2 Experiments with Profile F1 318
Figure 3.4.17  Summary of Maximum Displacements as Functions of Excavation Depth for Group G2 Experiments with Profile F2 319
Figure 3.4.18  Summary of Maximum Displacements as Functions of Excavation Depth for Group G2 Experiments with Profile F3 320
Figure 3.4.19  Summary of Maximum Displacements as Functions of Excavation Depth for Group G2 Experiments with Profile F4 321
Figure 3.4.20  Summary of Maximum Displacements as Functions
of the Bending Stiffness of the Support Wall for Group G2 Experiments

Figure 3.4.21  Moment Distribution in the Wall for Group G2 Experiments with Soil Profile F1 at H = 10-m

Figure 3.4.22  Moment Distribution in the Wall for Group G2 Experiments with Soil Profile F2 at H = 10-m

Figure 3.4.23  Moment Distribution in the Wall for Group G2 Experiments with Soil Profile F3 at H = 10-m

Figure 3.4.24  Moment Distribution in the Wall for Group G2 Experiments with Soil Profile F4 at H = 10-m

Figure 3.4.25a  Summary of Strut Forces for Group G2 Experiments with Profile F1(Struts 1 - 4)

Figure 3.4.25b  Summary of Strut Forces for Group G2 Experiments with Profile F1(Struts 5 - 8)

Figure 3.4.26a  Summary of Strut Forces for Group G2 Experiments with Profile F2 (Struts 1 - 4)

Figure 3.4.26b  Summary of Strut Forces for Group G2 Experiments with Profile F2 (Struts 5 - 8)

Figure 3.4.27a  Summary of Strut Forces for Group G2 Experiments with Profile F3 (Struts 1 - 4)

Figure 3.4.27b  Summary of Strut Forces for Group G2 Experiments with Profile F3 (Struts 5 - 8)

Figure 3.4.28a  Summary of Strut Forces for Group G2 Experiments with Profile F4 (Struts 1 - 4)

Figure 3.4.28b  Summary of Strut Forces for Group G2 Experiments with Profile F4 (Struts 5 - 6)

Figure 3.4.29  Group H Numerical Experiments for the Evaluation of the Strut Stiffness

Figure 3.4.30  Group H1 Numerical Experiments for the Evaluation of the Effects of Wall Support Stiffness

Figure 3.4.31  Wall Deflections and Surface Settlements for Group H1 Experiments at H = 5-m, 7.5-m and 10-m

Figure 3.4.32  Wall Deflections and Surface Settlements for Group H1 Experiments at H = 12.5-m, 15-m and 17.5-m

Figure 3.4.33  Summary of Maximum Displacements as Functions of Excavation Depth for Group H1 Experiments

Figure 3.4.34  Summary of Maximum Displacements as Functions of the Wall Support Stiffness (k) for Group H1 Experiments

Figure 3.4.35  Moment Distribution in the Wall for Group H1 Experiments

Figure 3.4.36a  Summary of the Strut Forces for Group H1 Experiments (Struts 1 - 4)

Figure 3.4.36b  Summary of the Strut Forces for Group H1 Experiments (Struts 5 - 7)

Figure 3.4.37  Group H2 Numerical Experiments for the Evaluation
of the Strut Stiffness (DW with rigid vs. real struts)

Figure 3.4.38  Wall Deflections and Surface Settlements for the Group H2 Experiments with Constant OCR = 1 Clay Profile 346

Figure 3.4.39  Wall Deflections and Surface Settlements for Group H2 Experiments with Profile F1 347

Figure 3.4.40  Wall Deflections and Surface Settlements for Group H2 Experiments with Profile F2 348

Figure 3.4.41  Wall Deflections and Surface Settlements for Group H2 Experiments with Profile F3 349

Figure 3.4.42  Wall Deflections and Surface Settlements for Group H2 Experiments with Profile F4 350

Figure 3.4.43  Summary of Maximum Displacements as Functions of Excavation Depth for Group H2 Experiments 351

Figure 3.4.44  Moment Distribution in the Wall for Group H2 Experiments 352

Figure 3.4.45a  Summary of Strut Forces for Group H2 Experiments (Struts 1 to 4) 353

Figure 3.4.45b  Summary of Strut Forces for Group H2 Experiments (Struts 5 to 8) 354

Figure 3.4.46  Group H3 Numerical Experiments for the Evaluation of the Strut Stiffness (SPW) with Rigid vs. Real Struts) 355

Figure 3.4.47  Wall Deflections and Surface Settlements for Group H3 Experiments with OCR = 1 Clay 356

Figure 3.4.48  Wall Deflections and Surface Settlements for Group H3 Experiments with Profile F1 357

Figure 3.4.49  Wall Deflections and Surface Settlements for Group H3 Experiments with Profile F2 358

Figure 3.4.50  Wall Deflections and Surface Settlements for Group H3 Experiments with Profile F3 359

Figure 3.4.51  Wall Deflections and Surface Settlements for Group H3 Experiments with Profile F4 360

Figure 3.4.52  Summary of Maximum Displacements as Functions of Excavation Depth for Group H3 Experiments 361

Figure 3.4.53  Moment Distribution in the Wall for Group H3 Experiments 362

Figure 3.4.54a  Strut Forces for Group H3 Experiments (Struts 1 - 4) 363

Figure 3.4.54b  Strut Forces for Group H3 Experiments (Struts 5 - 8) 364

Figure 4.1.1  Locations of the three Excavations in Boston: South Cove, South Boston, and MBTA Transitway 365

Figure 4.1.2  South Cove Project Layout [Jaworski, 1973] 366

Figure 4.1.3  Cross-section at South Cove Section A-A, Station 113+40 [Jaworski, 1973] 367

Figure 4.1.4  Cross-section at Section B-B, Station 111+40, South Cove Project [Jaworski, 1973] 368

Figure 4.1.5  Soil Profile and In-Situ Stress Conditions at South Cove, Section A-A 369
Figure 4.1.6  Finite Element Representation of Excavation History for South Cove, Section A-A  474
Figure 4.1.7  Construction Schedule at South Cove, Section A-A  475
Figure 4.1.8  Construction Schedule at South Cove, Section B-B  476
Figure 4.1.9  Finite Element Representation of Excavation History for South Cove, Section B-B  477
Figure 4.1.10  Finite Element Model for South Cove Section A-A  478
Figure 4.1.11  Simulation of the Installation and Prestress of Cross-Lot Bracing  479
Figure 4.1.12  Comparison of Predicted and Measured Wall Deflections and Surface Settlements for South Cove Section A-A at Excavation Step 2  480
Figure 4.1.13  Comparison of Predicted and Measured Wall Deflections and Surface Settlements for South Cove Section A-A at Excavation Step 6  481
Figure 4.1.14  Comparison of Predicted and Measured Wall Deflections and Surface Settlements for South Cove Section A-A at Excavation Step 7  482
Figure 4.1.15  Comparison of Predicted and Measured Wall Deflections and Surface Settlements for South Cove Section A-A at Excavation Step 8  483
Figure 4.1.16  Comparison of Predicted and Measured Pore Pressures for South Cove, Section A-A  484
Figure 4.1.17  Comparison of Predicted and Measured Strut Loads for South Cove, Section A-A  485
Figure 4.1.18  Comparison of Predicted and Measured Wall Deflections for South Cove Section B-B at Excavation Step 5  486
Figure 4.1.19  Comparison of Predicted and Measured Wall Deflections for South Cove Section B-B at Excavation Step 6  487
Figure 4.1.20  Comparison of Predicted and Measured Wall Deflections for South Cove Section B-B at Excavation Step 8  488
Figure 4.1.21  Comparison of Predicted and Measured Wall Deflections for South Cove Section B-B at Excavation Step 9  489
Figure 4.1.22  Comparison of Predicted and Measured Wall Deflections for South Cove Section B-B at Excavation Step 11  490
Figure 4.1.23  Comparison of Predicted and Measured Pore Pressures for South Cove Section B-B  491
Figure 4.2.1  Plan View of the Excavation at South Boston: Positions of Geotechnical Instrumentation and Instrumented Sections [Whelan, 1995, p. 87]  493
Figure 4.2.2  Plan View of ISS-4 Area: Locations of Building A and Geotechnical Instrumentation [Whelan, 1995, p. 145]  494
Figure 4.2.3  Cross-Sectional View of Excavation at ISS-4 Section (approximately at Station 77+20): Locations of tiebacks, Support Walls, and Geotechnical Instrumentation [Whelan, 1995, p. 144]  495
Figure 4.2.4 Dimensions and Engineering Properties of Arbed AZ-18 Sheet Piling [ISPC, 1990] (section used for South Wall) 496

Figure 4.2.5a Selected Effective Stress History Profile at ISS-4 Determined by MIT based on Re-evaluation of the Consolidation Test Data on BBC [Whelan, 1995, p. 130] 497

Figure 4.2.5b OCR and Undrained Strength Profiles for Marine Clay throughout the Project Alignment. Compiled from MHD Geotechnical Consultants (1991b) and Special Test Program (MHD Geotechnical Consultant, 1994) [Whelan, 1995, p. 129] 498

Figure 4.2.6 Initial In-Situ Stresses Throughout the ISS-4 Soil Profile [Whelan, 1995, p. 127] 499

Figure 4.2.7a Time Period Summary for Step 1: Excavation Geometry on May 7, 1993 [Whelan, 1995, p. 192] 500

Figure 4.2.7b Time Period Summary for Step 2: Excavation Geometry on June 29, 1993 [Whelan, 1995, p. 192] 501

Figure 4.2.7c Time Period Summary for Step 3: Excavation Geometry on July 9, 1993 [Whelan, 1995, p. 192] 502

Figure 4.2.7d Time Period Summary for Step 4: Excavation Geometry on August 9, 1993 [Whelan, 1995, p. 192] 503

Figure 4.2.7e Time Period Summary for Step 5: Excavation Geometry on September 2, 1993 [Whelan, 1995, p. 192] 504

Figure 4.2.7f Time Period Summary for Step 6: Excavation Geometry on October 6, 1993 [Whelan, 1995, p. 192] 505

Figure 4.2.7g Time Period Summary for Step 7: Excavation Geometry on March 11, 1994 [Whelan, 1995, p. 192] 506

Figure 4.2.7h Time Period Summary for Step 8: Excavation Geometry on May 31, 1994 [Whelan, 1995, p. 192] 507

Figure 4.2.7i Excavation Schedule as Documented by Whelan [1995] at South Boston ISS-4 508

Figure 4.2.8 Locations of Dewatering and Pressure Relief Wells [Whelan, 1995, p. 70] 509

Figure 4.2.9 Piezometric Elevations in the Lower Aquifer vs. Time Measured by Four Deep Piezometers Located below the BBC [Whelan, 1995, p. 180] 510

Figure 4.2.10 Finite Element Mesh used for the Analysis of Excavation at South Boston ISS-4 511

Figure 4.2.11a Construction Sequence Incorporated in the Base Case Analysis at Steps 1S, 1N, and 2 512

Figure 4.2.11b Construction Sequence Incorporated in the Base Case Analysis at Steps 3, 4, and 5 513

Figure 4.2.11c Construction Sequence Incorporated in the Base Case Analysis at Steps 6 and 7 514

Figure 4.2.12 Piezometric Elevations vs. Time, Measured by Four Deep Piezometers in Lower Aquifer vs. Imposed
Figure 4.2.13  Piezometric Elevations
Wall Deflections and Surface Settlements at Construction
Step 1S [May 7 to June 5, 1993], Base Case Analysis versus
Field Measurements

Figure 4.2.14  Wall Deflections and Surface Settlements at Construction
Step 1N [Mar. 22 to May 7, 1993], Base Case Analysis versus
Field Measurements

Figure 4.2.15  Wall Deflections and Surface Settlements at Construction
Step 2 [June 5 to July 1, 1993], Base Case Analysis versus
Field Measurements

Figure 4.2.16  Wall Deflections and Surface Settlements at Construction
Step 3 [July 1 to Aug. 9, 1993], Base Case Analysis versus
Field Measurements

Figure 4.2.17  Wall Deflections and Surface Settlements at Construction
Step 4 [Aug. 9 to Aug. 20, 1993], Base Case Analysis versus
Field Measurements

Figure 4.2.18  Wall Deflections and Surface Settlements at Construction
Step 5 [Aug. 20 to Sept. 15, 1993], Base Case Analysis versus
Field Measurements

Figure 4.2.19  Wall Deflections and Surface Settlements at Construction
Step 6 [Sept. 15, 1993 to Mar. 2, 1994], Base Case Analysis versus
Field Measurements

Figure 4.2.20  Wall Deflections and Surface Settlements at Construction
Step 7 [Mar. 4 to Mar. 11, 1994], Base Case Analysis versus
Field Measurements

Figure 4.2.21  Piezometric Elevations vs. Time, Measured by OW-016
and OW-002 (North Side of Excavation) vs. Base Case
Analysis

Figure 4.2.22  Piezometric Elevations vs. Time, Measured by VWPZ-67
and VWPZ-68 (North Side of Excavation) vs. Base Case
Analysis

Figure 4.2.23  Piezometric Elevations vs. Time, Measured by OSPZ-106
(South Side of Excavation) vs. Base Case Analysis

Figure 4.2.24  Piezometric Elevations vs. Time, Measured by VWPZ-135
and VWPZ-136 (Inside of Excavation) vs. Base Case
Analysis

Figure 4.2.25  Piezometric Elevations vs. Time, Measured by VWPZ-133
and VWPZ-134 (Inside of Excavation) vs. Base Case
Analysis

Figure 4.2.26  Piezometric Elevations vs. Time, Measured by VWPZ-131
and VWPZ-132 (Inside of Excavation) vs. Base Case
Analysis

Figure 4.2.27  Heave in Clay vs. Time, Measured by MPHG-110
(Inside of Excavation) vs. Base Case Analysis

Figure 4.2.28  Heave in Clay vs. Time, Measured by MPHG-109
(Inside of Excavation) vs. Base Case Analysis
Figure 4.2.29  Heave in Clay vs. Time, Measured by MPHG-501 (Inside of Excavation) vs. Base Case Analysis 532

Figure 4.2.30  Heave in Clay vs. Time, Measured by MPHG-107 (Inside of Excavation) vs. Base Case Analysis 533

Figure 4.2.31  Wall Deflections vs. Time, Measured by INC-102 (North Diaphragm Wall) vs. Base Case Analysis 534

Figure 4.2.32  Wall Deflections vs. Time, Measured by INC-101 (South Sheetpile Wall) vs. Base Case Analysis 535

Figure 4.2.33  Wall Deflections vs. Time, Measured by IPE-113 (behind South Sheetpile Wall) vs. Base Case Analysis 536

Figure 4.2.34  Surface Settlements vs. Time, Measured at Settlement Points behind North Diaphragm Wall vs. Base Case Analysis 537

Figure 4.2.35  Surface Settlements vs. Time, Measured at Settlement Points behind South Sheetpile Wall vs. Base Case Analysis 538

Figure 4.2.36  Settlements vs. Time, Measured at IPE-113 (behind South Sheetpile Wall) vs. Base Case Analysis 539

Figure 4.2.37  Revised Case FE Analysis including Effects of (1) Underlying Till and Bedrock; (2) Revised Permeability; and (3) Omission of 2-S Tiebacks 540

Figure 4.2.38  Wall Deflections and Surface Settlements at Construction Step 1N [Mar. 22 to May 7, 1993], Revised Case Analysis versus Field Measurements 541

Figure 4.2.39  Wall Deflections and Surface Settlements at Construction Step 2 [June 5 to July 1, 1993], Revised Case Analysis versus Field Measurements 542

Figure 4.2.40  Wall Deflections and Surface Settlements at Construction Step 3 [July 1 to Aug. 9, 1993], Revised Case Analysis versus Field Measurements 543

Figure 4.2.41  Wall Deflections and Surface Settlements at Construction Step 4 [Aug. 9 to Aug. 20, 1993], Revised Case Analysis versus Field Measurements 544

Figure 4.2.42  Wall Deflections and Surface Settlements at Construction Step 5 [Aug. 20 to Sept. 15, 1993], Revised Case Analysis versus Field Measurements 545

Figure 4.2.43  Wall Deflections and Surface Settlements at Construction Step 6 [Sept. 15, 1993 to Mar. 2, 1994], Revised Case Analysis versus Field Measurements 546

Figure 4.2.44  Wall Deflections and Surface Settlements at Construction Steps 1N, 3, 5, and 7 Predicted by the Revised Case and Revised Case Analyses 547

Figure 4.2.45  Piezometric Elevations vs. Time, Measured by OW-016 and OW-002 (North Side of Excavation) vs. Revised Case Analysis 548

Figure 4.2.46  Piezometric Elevations vs. Time, Measured by VWZPZ-67 and VWZPZ-68 (North Side of Excavation) vs. Revised Case Analysis 549
Figure 4.2.47 Piezometric Elevations vs. Time, Measured by OSPZ-106 (South Side of Excavation) vs. Revised Case Analysis 550
Figure 4.2.48 Piezometric Elevations vs. Time, Measured by VWPZ-135 and VWPZ-136 (Inside of Excavation) vs. Revised Case Analysis 551
Figure 4.2.49 Piezometric Elevations vs. Time, Measured by VWPZ-133 and VWPZ-134 (Inside of Excavation) vs. Revised Case Analysis 552
Figure 4.2.50 Piezometric Elevations vs. Time, Measured by VWPZ-131 and VWPZ-132 (Inside of Excavation) vs. Revised Case Analysis 553
Figure 4.2.51 Heave in Clay vs. Time, Measured by MPHG-110 (Inside of Excavation) vs. Revised Case Analysis 554
Figure 4.2.52 Heave in Clay vs. Time, Measured by MPHG-109 (Inside of Excavation) vs. Revised Case Analysis 555
Figure 4.2.53 Heave in Clay vs. Time, Measured by MPHG-501 (Inside of Excavation) vs. Revised Case Analysis 556
Figure 4.2.54 Heave in Clay vs. Time, Measured by MPHG-107 (Inside of Excavation) vs. Revised Case Analysis 557
Figure 4.2.55 Wall Deflections vs. Time, Measured by INC-102 (North Diaphragm Wall) vs. Revised Case Analysis 558
Figure 4.2.56 Wall Deflections vs. Time, Measured by INC-101 (South Sheetpile Wall) vs. Revised Case Analysis 559
Figure 4.2.57 Wall Deflections vs. Time, Measured by IPE-113 (behind South Sheetpile Wall) vs. Revised Case Analysis 560
Figure 4.2.58 Surface Settlements vs. Time, Measured at Settlement Points behind North Diaphragm Wall vs. Revised Case Analysis 561
Figure 4.2.59 Surface Settlements vs. Time, Measured at Settlement Points behind South Sheetpile Wall vs. Revised Case Analysis 562
Figure 4.2.60 Settlements vs. Time, Measured at IPE-113 (behind South Sheetpile Wall) vs. Revised Case Analysis 563
Figure 4.3.1 Location Plan of MBTA Transitway Project and South Boston Special Test Site 564
Figure 4.3.2 Stress History Profile for BBC, Based on Data from South Boston Special Test Site [Original Profile] 565
Figure 4.3.3 MIT-E3 Representation of Undrained Shear Strength for BBC [Original Profile] 566
Figure 4.3.4 Comparison of MIT-E3 [Original Profile] and SHANSEP Strength Profile for BBC 567
Figure 4.3.5 Comparison of Field Vane and Undrained DSS Strength for Original Profile [Revised Profile] 568
Figure 4.3.6 Comparison of MIT-E3 [Revised Profile] and SHANSEP Strength Profiles for BBC 569
Figure 4.3.7 Soil Profile, In-situ Stresses, and Pore Pressures at the

Page 27
| Figure 4.3.8 | Excavation Sequence for Platform Section with Excavation Geometry P1 \( [h = 10\text{-ft}, \text{toe of DW @ El. 24-ft}] \) |
| Figure 4.3.9 | Excavation Sequence for Platform Section with Excavation Geometry P2 \( [h = 13\text{-ft}, \text{toe of DW @ El. 24-ft}] \) |
| Figure 4.3.10 | Excavation Sequence for Platform Section with Excavation Geometry P3 \( [h = 13\text{-ft}, \text{toe of DW @ El. 34-ft}] \) |
| Figure 4.3.11 | Excavation Sequence for the Transitway Mezzanine Section (M) |
| Figure 4.3.12 | Excavation Sequence for the Transitway Transition Section (T) |
| Figure 4.3.13 | Excavation Sequence for the Transitway West Tunnel Section (W1) |
| Figure 4.3.14 | Typical Finite Element Mesh used to Model MBTA Transitway Excavation |
| Figure 4.3.15 | Wall Displacement for Transitway Platform Section, Excavation Geometry P1, with Original Profile and Undrained Analysis [P1-O-UD] |
| Figure 4.3.16 | Ground Surface Displacements for the Transitway Platform Section, Excavation Geometry P1, with Original Profile and Undrained Analysis [P1-O-UD] |
| Figure 4.3.17 | Bending Moments in the Diaphragm Wall for Transitway Platform Section, Excavation Geometry P1, with Original Profile and Undrained Analysis [P1-O-UD] |
| Figure 4.3.18 | Wall Displacement for Transitway Platform Section, Excavation Geometry P2, with Original Profile and Undrained Analysis [P2-O-UD] |
| Figure 4.3.19 | Ground Surface Displacements for the Transitway Platform Section, Excavation Geometry P2, with Original Profile and Undrained Analysis [P2-O-UD] |
| Figure 4.3.20 | Bending Moments in the Diaphragm Wall for Transitway Platform Section, Excavation Geometry P2, with Original Profile and Undrained Analysis [P2-O-UD] |
| Figure 4.3.21 | Maximum Wall Deflection, Surface Settlement, and Surface Horizontal Displacement from P1-O-UD and P2-O-UD: Strut Spacing Effects |
| Figure 4.3.22 | Maximum Diaphragm Wall Bending Moments Above and Below the Lowest Level Strut from P1-O-UD and P2-O-UD: Strut Spacing Effects, Platform Section |
| Figure 4.3.23 | Strut Loads for Transitway Platform Section with Excavation Geometries P1 and P2, Undrained Analysis and Original Profile [P1-O-UD and P2-O-UD] |
| Figure 4.3.24 | Wall Deflections for the Transitway Platform Section, Excavation Geometry P3 with Original Profile and Undrained Analysis [P3-O-UD] |
| Figure 4.3.25 | Ground Surface Displacements for the Transitway |
Platform Section, Excavation Geometry P3, with Original Profile and Undrained Analysis [P3-O-UD]

Figure 4.3.26 Bending Moments in the Diaphragm Wall for Transitway Platform Section, Excavation Geometry P3, with Original Profile and Undrained Analysis [P3-O-UD]

Figure 4.3.27 Maximum Wall Deflection, Surface Settlement, and Surface Horizontal Displacement from P2-O-UD and P3-O-UD: Wall Length Effects

Figure 4.3.28 Maximum Diaphragm Wall Bending Moments Above and Below the Lowest Level Strut from P2-O-UD and P3-O-UD: Wall Length Effects, Platform Section

Figure 4.3.29 Strut Loads for Transitway Platform Section with Excavation Geometries P2 and P3, Undrained Analysis and Original Profile [P2-O-UD and P3-O-UD]

Figure 4.3.30 Wall Displacement for the Transitway Platform Section, Excavation Geometry P1, with Original Profile and Partially Drained Analysis [P1-O-PD]

Figure 4.3.31 Ground Surface Displacements for the Transitway Platform Section, Excavation Geometry P1, with Original Profile and Partially Drained Analysis [P1-O-PD]

Figure 4.3.32 Bending Moments in the Diaphragm Wall for Transitway Platform Section, Excavation Geometry P1, with Original Profile and Partially Drained Analysis [P1-O-PD]

Figure 4.3.33 Maximum Wall Deflection, Surface Settlement, and Surface Horizontal Displacements form P1-O-UD and P1-O-PD: Partial Drainage Effects

Figure 4.3.34 Maximum Diaphragm Wall Bending Moments Above and Below the Lowest Level Strut from P1-O-UD and P1-O-PD: Partial Drainage Effects, Platform Section

Figure 4.3.35 Strut Loads for Transitway Platform Section with Excavation Geometries P1, P1-O-UD and P1-O-PD: Partial Drainage Effects

Figure 4.3.36 Wall Displacement for the Transitway Platform Section, Excavation Geometry P3 with Original Profile and Partially Drained Analysis [P3-O-PD]

Figure 4.3.37 Ground Surface Displacements for the Transitway Platform Section, Excavation Geometry P3, with Original Profile and Partially Drained Analysis [P3-O-PD]

Figure 4.3.38 Bending Moments in the Diaphragm Wall for Transitway Platform Section, Excavation Geometry P3, with Original Profile and Partially Drained Analysis [P3-O-PD]

Figure 4.3.39 Maximum Wall Deflections, Surface Settlements, and Surface Horizontal Displacements from P3-O-UD and P3-O-PD: Partial Drainage Effects

Figure 4.3.40 Maximum Diaphragm Wall Bending Moments Above and Below the Lowest Level Strut from P3-O-UD and
Revised Profiles and Partially Drained Analysis
[T1-O-PD and T1-R-PD] 619

Figure 4.3.57  Ground Surface Displacements for the Transitway Transition Section, with Original and Revised Profiles and Partially Drained Analysis [T1-O-PD and T1-R-PD] 620

Figure 4.3.58  Maximum Wall Deflection, Surface Settlement, and Surface Horizontal Displacement from T1-O-PD and T1-R-PD: Transition Section Analysis Summary 621

Figure 4.3.59  Maximum Diaphragm Wall Bending Moments Above and Below the Lowest Level Strut from T1-O-PD and T1-R-PD: Transition Section 622

Figure 4.3.60  Strut Loads for Transitway Transition Section with Original and Revised Profiles, Partially Drained Analysis [T1-O-PD and T1-R-PD] 623

Figure 4.3.61  Transitway West Tunnel Section Cross-Sectional View 624

Figure 4.3.62  Wall Displacement and Moment in the Diaphragm Wall for the West Tunnel Section with Original and Revised Profiles and Partially Drained Analysis [W1-O-PD and W1-R-PD] 625

Figure 4.3.63  Ground Surface Displacements for the Transitway West Tunnel Section, with Original and Revised Profiles and Partially Drained Analysis [W1-O-PD and W1-R-PD] 626

Figure 4.3.64  Maximum Wall Deflection, Surface Settlement, and Surface Horizontal Displacement from W1-O-PD and W1-R-PD: West Tunnel Section Analysis Summary 627

Figure 4.3.65  Maximum Diaphragm Wall Bending Moments Above and Below the Lowest Level Strut from W1-O-PD and W1-R-UD: West Tunnel Section 628

Figure 4.3.66  Strut Loads for Transitway West Tunnel Section with Original and Revised Profiles, Partially Drained Analysis [W1-O-PD and W1-R-PD] 629

Figure 4.3.67  Summary of Maximum Displacements from the Four Typical Sections in the Transitway Project: Width Effects 630

Figure 5.1.1  Settlement Trough Characteristics for Excavations in Normally Consolidated BBC 649

Figure 5.1.2  Equation for Describing Settlement Troughs for Undrained Excavations in Normally Consolidated BBC 649

Figure 5.1.3  Calculation of Coefficients for Equation 5.1.2 650

Figure 5.1.4  Definition of Adjusted Depth to Bedrock Parameter, \( d^* \) \(_B\) 651

Figure 5.1.5  Procedure for Determining the Distribution of Surface Settlements for Excavations in Normally Consolidated Boston Blue Clay 652

Figure 5.1.6a  Evaluation of Settlement Predictions for Excavations in Normally Consolidated BBC [\( B = 80\text{-m} \)] 653
Figure 5.1.6b  Evaluation of Settlement Predictions for Excavations in Normally Consolidated BBC [B = 40-m]  654
Figure 5.1.6c  Evaluation of Settlement Predictions for Excavations in Normally Consolidated BBC [B = 20-m]  655
Figure 5.1.7  Predicted and Actual Maximum Settlements for Undrained Excavations in Normally Consolidated BBC  656
Figure 5.1.8  Prediction of Maximum Surface Settlement for Excavations in Normally Consolidated BBC  657
Figure 5.1.9a  Illustration of the Predictive Capabilities for Estimating Settlement for Undrained Excavations in Normally Consolidated BBC [L = 25-m]  658
Figure 5.1.9b  Illustration of the Predictive Capabilities for Estimating Settlement for Undrained Excavations in Normally Consolidated BBC [L = 40-m]  659
Figure 5.2.1  Estimation of Maximum Surface Settelsments in Overconsolidated BBC  661
Figure 5.2.2  Adjustment Factor for Overlying Cohesionless Soil  662
Figure 5.2.3  Recommendation for Estimating Maximum Surface Settlements in Composite Soil Profiles  663
Figure 5.2.4  Method for Estimating the Distribution of Surface Settlement in Composite Profile  664
Figure 5.2.5a  Illustration of Prediction Capabilities for Composite Soil Profile F1  665
Figure 5.2.5b  Illustration of Prediction Capabilities for Composite Soil Profile F3  666
Figure 5.2.5c  Illustration of Prediction Capabilities for Composite Soil Profile F4  667
Figure 5.3.1  Effects of Support Stiffness on Maximum Surface Settlements for Excavations in Normally Consolidated BBC  669
Figure 5.3.2a  Illustration of Proposed Support System Stiffness Adjustments for Excavations in Profiles F1 and F2  670
Figure 5.3.2b  Illustration of Proposed Support System Stiffness Adjustments for Excavations in Profiles F3 and F4  671
Figure 5.3.3  Calculation of Support System Stiffness Adjustment Factor, \( \pi \)  672
Figure 5.3.4  Recommended Method for Prediction Surface Settlements in Normally Consolidated BBC  673
Figure 5.3.5  Recommended Method for Predicting Surface Settlements for Composite Profiles  674
Figure A1  Comparison of Input Definition for Excess Pore Pressure (V4.9) versus Total Pore Pressure (V5.4) Formulations  694
Figure A2  Comparison of Excavation Procedure used in ABAQUS V4.9 versus ABAQUS V5.4  695
Figure A3  Modifications in User Subroutine UMAT and SIGINI  696
Figure A4  Excavation Procedure for Using User Elements (UEL)  697
CHAPTER 1

Introduction

In order to accommodate both population and economic growth, repair, upgrading, and expansion of the existing infrastructure within major metropolitan areas is inevitable. Given that open areas are rather scarce, many cities have opted to utilize subsurface space by replacing surface infrastructures such as roadways, garages, and mass transit systems with underground facilities. This quest for space has led to numerous innovative construction methods with the goals of reducing costs, disruption of existing services, and damages to adjacent structures. For underground construction in areas with underlying soft clay deposits, controlling, or even predicting ground deformations due to excavations is a difficult task.

Most of the existing design recommendations are based on excavations supported by sheetpile walls; unfortunately, guidelines for predicting deformations due to deep excavations in soft clay supported by more stiffer walls are rather limited. Approximate design methods for predicting soil deformations and loads on structural system may be adequate in situations where these parameters are not critical in design (i.e. stiff underlying soil or no sensitive adjacent structures). These existing methods are generally based
on observations of past constructions and are not capable of generating reliable and accurate predictions of ground deformations for cases that do not possess the same characteristics. Consequently, these empirically-based recommendations simply summarize the overall performance of selected past excavations without quantifying the isolated effects of the various factors nor providing better understanding of ground movements in response to excavations. This leads to the general focus of this research: developing improved methods for predicting excavation-induced displacements and guidelines for the design of supporting structures. These design recommendations are developed using non-linear finite element analyses which incorporate a generalized effective stress soil model, MIT-E3 (Whittle, 1987), capable of capturing the anisotropic stress-strain-strength, small strain non-linearity, and hysteretic and path dependent behavior of normally consolidated to lightly overconsolidated clays.

With the ultimate goal of developing design recommendations for deep excavations in clay, this dissertation is divided into four sections: (1) expansion of the current numerical analysis capabilities; (2) an extensive parametric study focusing on the effects of excavation geometry, soil profile, and support system; (3) evaluation of two case histories and one design study in the Boston area; and (4) development of design recommendations for predicting ground deformations.

1.1 Expansion of Numerical Techniques

The first section of the thesis, Chapter 2, focuses on defining the various components affecting excavation performance and describing existing methods which attempt to quantify these factors. The second half of Chapter 2 explains the numerical techniques used in this investigation to evaluate the
isolated effects of selected factors for the purpose of providing better understanding of the behavior of excavations. The basic numerical technique was developed by Hashash [1992] as a part of his doctoral research. He incorporated the MIT-E3 soil model [Whittle, 1987] in a commercially available finite element program ABAQUS™ and demonstrated this is a promising technique for the analysis of excavation especially in soft clay [Hashash, 1992]. In addition to describing the features in the MIT-E3 model and the implementation of MIT-E3 in ABAQUS, Chapter 2 also includes descriptions of additional features added in the course of this research which expands the numerical capability. The added features include implementation of (1) an user-defined 15-3 triangular element capable of modeling undrained axisymmetric non-linear analyses; and (2) for plane strain analyses, numerical procedures for representing prestressed tieback anchors in clay.

1.2 Parametric Study

Chapter 3 consists of three main groups of parametric analyses to quantify the effects of excavation geometry, soil profile, and support system. This parametric study consists of performing pre-defined groups of undrained plane strain numerical excavation analyses in soft clay for the purpose of determining the influence of each factor. The performance of the excavation is evaluated on the basis of the resulting wall deflections, surface movements, strut forces, and moments in the support wall. The first section reviews the findings from previous research [Hashash and Whittle, 1993]. The remaining sections examine the influence of (I) excavation geometry, (II) soil profile, and (III) support system.
The first set of parametric analyses focuses on the effects of excavation geometry. The evaluation of excavation geometry consists of examining the impact of wall length (L), excavation width (B), and depth to bedrock (dB). Particular attention is given to the resulting settlement trough as damage to adjacent structures is closely related to the surface deformations. The experiments as well as results are presented in Section 3.2.

The second set of parametric analyses focuses on how variations in soil profile impact the excavation performance. Three types of variations are considered: (a) the overconsolidation ratio (OCR) of the clay, (b) a cohesionless soil layer overlying the clay stratum, and (c) presence of a clay crust overlying a low OCR clay stratum. The results of these numerical experiments are presented in Section 3.3.

The last set of parametric analyses examines the role of supporting structures in controlling excavation behavior. The support system consists of two components: the support wall and the wall supports. The numerical analyses focus on the influence of the stiffness of the support wall and the stiffness of the wall supports. These analyses also evaluate the effectiveness of the support conditions as a function of the soil profile. These analyses do not consider the potential installation effects of the wall and the supports.

1.3 Case Studies

Analyses of three excavations in the Boston area are presented and analyzed in Chapter 4. The first two case studies, South Cove (a cross-lot braced excavation) and South Boston (CA/T) Section ISS4 (a wide open cut with tieback anchor support) have been completed and actual field measurements are used to assess the performance of the numerical analyses.
The third case, the MBTA Transitway, is still in the design stage and the numerical results are used to evaluate the proposed design.

1.4 Development of Design Recommendations

Based on the results of the parametric analyses and the case studies, the primary goal of this research of providing improved design recommendations is presented in Chapter 5. The parametric analyses provide insights on how the soil responds to the excavation process and changes in specific factors; the case studies illustrate the limitations and capabilities of this numerical technique. The intent of these design recommendations is to provide reliable predictions of the magnitude and distribution of surface settlements. Summary, conclusions, and recommendations for future research are presented in Chapter 6.
Development of Finite Element Capabilities
for Analyzing Excavations in Clay

Conventional methods for designing excavation support systems generally consist of three criteria. The first two criteria consider the two major categories of failure modes: (1) yielding or collapse of the support system (including both the wall and the bracing system) and (2) stability against bottom heave and piping1. In addition to avoiding failures of the excavation itself, the excavation support system must also prevent excessive excavation-induced damage to adjacent structures. From the design perspective, this "allowable level of damage" is usually expressed in terms of tolerable surface settlements and lateral movements. Consequently, the overall design for an excavation support system also include this third criterion: limited ground movements.

In most urban settings, where damage to the existing structures is the utmost concern, the third criterion of limited movements is the primary design consideration and constraint. Therefore, the main focus of this thesis

---

1 Excessive bottom heave is generally a design consideration in cohesive soils, while piping (or quick condition) is major concern in free-draining granular soils.
is on developing improved methods of predicting soil movements in response to excavations. The purpose of this chapter is to provide (1) an overview of the various components which affect excavation performance, (2) a summary of existing methods which attempt to quantify these relationships, and (3) a detailed description of the proposed method for analyzing excavations.

Section 2.1 briefly describes the various factors which influence the behavior of excavations in clay and Section 2.2 summarizes the most commonly used design methods. Existing empirical methods for predicting excavation-induced movements are only capable of giving very approximate estimates of ground movements, and give not insight into factors affecting their magnitude and distribution. More reliable site specific calculations can be achieved using non-linear finite element analyses which incorporate realistic constitutive models of soil behavior. For design purposes, results of parametric finite element calculations can then be interpreted for generic prediction of ground movements. This thesis uses the MIT-E3 soil model [Whittle, 1987] to represent soft Boston Blue Clay. This model was integrated in the ABAQUS finite element code\(^2\) by Hashash [1992] as a part of his doctoral research. Brief descriptions of this numerical implementation, as well as the MIT-E3 soil model developed by Whittle [1987], are presented in Section 2.3. Section 2.4 focuses on the further development of numerical analysis capabilities undertaken in the current research program. These advancements include modifications in the excavation procedure using ABAQUS, implementation of special high order finite elements (15-3

\(^2\) ABAQUS is a commercially available general purpose finite element program written and distributed by Hibbitt, Karlsson & Sorenson, Inc. (HKS). This program is made available to MIT under an academic license agreement. ABAQUS Version 5.4 was used for all the analyses presented in this thesis.
elements\textsuperscript{3}) to simulate undrained axisymmetric problems, and modeling of the various components of the support systems.

2.1 Factors Influencing Excavation Performance in Clay

In most excavation-induced deformation studies, factors which affect ground movements are generally expressed in terms of specific construction activities. Table 2.1.1, modified based on Mana and Clough [1981], summarizes these typically cited factors which include soil properties, stiffness of the wall, construction method. As shown in Table 2.1.1, these factors are classified into three categories based on the construction stage when these variables are defined: site conditions, design parameters, and construction parameters.

These basic components which determine the excavation performance are highly influenced by soil behavior at the element level. The behavior of a soil element is determined by the initial conditions, the stress-strain-strength characteristics of the soil, and the stress path that the soil element is subjected to. Likewise, the excavation behavior can also be defined by these three components.

Closer examination of the factors listed in Table 2.1.1 reveals that they can also be considered as one of the three components described above: initial conditions, stress-strain-strength characteristics, and changes in stress. Figure 2.1.1 illustrates an alternative and more general view of classifying these factors. As shown in the figure, factors that fall into the "changes in stress" component identify various means of controlling the stress changes in the soil and the other two components describe the existing conditions and

\textsuperscript{3} The 15-3 element is a triangular element consisting of 15 displacement nodes and 3 pore pressure nodes (see Section 2.4.2).
generally cannot be altered. The combination of the initial conditions, soil stress-strain-strength characteristics, and the changes in stress defines the excavation performance which is evaluated on the basis of soil movements.

As illustrated in Figure 2.1.1, the prediction of the excavation behavior is a forward process where the goal is to define the performance, $f(x_i)$, given that all the factors and corresponding effects, $x_i$'s, are known. The development of design charts, however, is a reverse process where certain factors, $x_i$'s, need to be defined in order to achieve a certain level of performance, $f(x_i)$. The different design methods used in practice arise from different means of capturing this relationship between the various factors and the performance, $\partial f(x_i)/\partial x_i$. The next section describes how some of these methods attempt to quantify these relationships.

2.2 Summary of Current Design Practices for Excavations in Clay

The existing design recommendations for excavations focus on two sets of parameters: prediction of ground movements and bracing forces. As mentioned previously, soil displacements are generally the limiting component in design, therefore, predicting the soil movements is usually the primary design consideration. Section 2.2.1 summarizes various methods of predicting the magnitude and distribution of soil movements in response to removal of soil. The second design issue is to ensure adequate structural design of the bracing system based on the maximum estimated forces (prestressing of the bracing can also be used to pro-actively control ground movements). Section 2.2.2 presents a brief description of the more commonly adopted design recommendations for excavation supports.

---

4 Improvement techniques such as deep soil mixing can also be classified under the heading of altering the soil stress-strain-strength characteristics.
2.2.1 Prediction of Soil Movements

Currently, there are no standard design methods for estimating ground movements caused by deep excavations\(^5\). In general, existing methods of predicting excavation performance are either based on empirical observations or numerical modeling. Because of the inherent complexities in the staged excavation, to date, neither of the two approaches can satisfactorily account and quantify the influence of every factor which controls ground movements. Hence, the existing methods are still not capable of producing accurate and reliable estimates of ground deformations.

The influence of the individual factors cannot be extracted from the empirical database due to limited number of excavations in similar soil and construction conditions. In contrast, existing numerical solutions\(^6\) tend to be site-specific and not available to generalized design recommendations. Despite the inadequate performance in the existing methods, these two general approaches represent two viable routes in understanding the problem of excavation-induced deformations. With this in mind, studying the existing methods provide a good outline of what has been attempted to date and which facets need to be investigated further. The next two sub-sections focus on some of these empirical and numerical findings and recommendations.

---

5 In this thesis, a deep excavation refers to any excavation exceeding a depth of 6.0-m [19.7-ft].
6 The numerical solutions also contain approximations in modeling soils, estimation of initial conditions, etc. [see Whittle, et al., 1993].
2.2.1.1 Empirical Approach

Empirical studies attempt to develop general relationships between observed ground movements and construction activities based on actual observations from a number of similar excavations. The three most typically cited empirical methods for estimating excavation-induced ground movements are by Peck [1969], Mana and Clough [1981], and Clough et al. [1989].

Though all deep excavation projects, to some extent, contain ground movements information, not all excavation projects possess the necessary qualifications to be included in a empirical study. This may be due to a variety of reasons ranging from insufficient or inaccurate field measurements to restricted use of data due to possible litigation concerns. Even if the data are available for analysis, extracting these specific relationships requires detailed compilation of numerous cases including unique nature of each excavation. For these reasons, quantifying effects of individual factors using the empirical approach is not a trivial task.

The first major compilation of excavation field data was published by Peck [1969]. He compiled ground deformation measurements from excavations performed in San Francisco, Seattle, Chicago, St. Louis, and Oslo (Norway). All of these excavations were supported by either soldier piles and lagging or sheetpile walls. A variety of wall supports were included in the case studies ranging from cross-lot bracing, pre-stressed rakers, H-pile tiebacks, and anchors. In addition to the variations in wall support system, the excavations took place within wide range of soils -- soft to medium clay, stiff clay, cemented sand, and cohesionless sand. The excavation depths, $H$, range from 6-m to 19-m [20 to 63-ft]. With this database, Peck generated a summary of the expected normalized limits and normalized magnitudes of the
settlement troughs as a function of soil type, excavation depth, and "workmanship"; both the limits and the magnitudes are normalized with respect to the excavation depth, H (see Figure 2.2.1). His findings suggest that excavations within a thick layer of soft to medium clay can generate large settlements, often greater than 2% of the excavation depth adjacent to the support wall, and extend laterally up to four times the excavated depth from the wall. Considering that this chart is based on data from relatively flexible support walls, extrapolating the results for the much stiffer diaphragm wall is not reliable. Despite this shortcoming, the fact that this is the first comprehensive study of its kind, the results reported by Peck are virtually always cited when discussing excavation-induced ground movements.

Mana and Clough [1981] presented more recent analyses of observed movements for braced cuts in clay. Their data also focus on excavations supported by sheetpile walls or soldier piles and mostly with cross-lot bracing. A total of 11 case histories7 were included in their empirical study. The report a definite correlation between the measured normalized lateral wall deflection (δw/H) to the factor of safety against basal heave as defined by Terzaghi [1943] (Figure 2.2.2). For factors of safety against basal heave greater than 1.5, the expected maximum wall movement is in the range 0.2% ≤ δw/H ≤ 1.0%. However, much larger wall deflections can occur for deep excavations in soft clay where factors of safety against basal heave are usually less than 1.5. Uncertainties in the estimates of δw are generally much larger than are needed in design. Since damage to adjacent structures is related to surface settlements and horizontal displacements, Mana and Clough also attempted

---

7 Mana and Clough determined that these 11 cases are similar in that excavation stress relief was the primary cause for ground movements. Over 100 other cases were eliminated because the large deformations were caused by "unusual construction effects" such as consolidation due to dewatering.
to relate maximum lateral wall movement to maximum settlement. As shown in Figure 2.2.3, the maximum settlement, \( \delta_v \), is generally between 0.5 to 1.0 times the maximum horizontal wall displacements, \( \delta_w \). Similar to Peck's results, Mana and Clough's findings are based on observations from excavations supported by soldier piles or sheet piles, therefore, cannot be extrapolated to other support conditions easily. In addition to the empirical study, Mana and Clough also presented results of their numerical analyses. Figure 2.2.4 summarizes their recommendations of evaluating the effects of strut stiffness, depth to firm layer, and excavation width on maximum lateral wall movements and settlement.

Clough et al. [1989]\(^8\) updated the existing database by incorporating the performance of excavations supported by diaphragm walls and relating the deformations to the support conditions and soil profiles. They noted that when factor of safety against basal heave is less than 1.5, the system stiffness\(^9\) can significantly influence the soil movements (see Figure 2.2.5). The authors also recommend dimensionless settlement trough profile (\( \delta_v/\delta_{v(max)} \), where \( \delta_{v(max)} \) is extracted from Figures 2.2.2 and 2.2.3), for three different soil conditions: sands, stiff to very hard clays, and soft to medium clays as shown in Figures 2.2.6 and 2.2.7. The authors suggest that the envelope of normalized surface settlements, \( \delta_v/\delta_{v(max)} \), is related to factor of safety against basal heave (Figure 2.2.6). For FS < 1.5, their proposed design shows no settlement beyond twice the excavation depth (from the excavation).

The results from Mana and Clough [1981] and Clough et al. [1989] were incorporated into a commercially available computer program called MOVEX [Smith, 1987]. The input requirements consist of (1) the undrained strength

---

\(^8\) Also refer to Clough and O'Rourke, 1990.
\(^9\) Clough and O'Rourke define the system stiffness as a function of the wall bending stiffness, EI, and the wall support spacing, h.
profile, (2) stiffness for the wall and wall supports, and (3) the excavation dimensions including the excavation depth, width, length, and vertical spacing of the wall supports. The following calculations are performed within MOVEX:

1. Based on the undrained shear strength profile, the factor of safety against basal heave, FS, is calculated as

\[
FS = \frac{N_c \sum (s_{ub} \times t_i)}{\left[ \sum (\gamma \times h_i) + q T - \sum (s_{us} \times h_i) \right]}
\]

where
- \( s_{us} \): S \( s_u \) along the side of the excavation above the excavation grade
- \( s_{ub} \): \( s_u \) between the excavation grade and depth \( T \) below the excavation grade
- \( \gamma \): total unit weight of the soil along side of excavation above the excavation grade
- \( q \): surcharge load
- \( N_c \): \( 5(1+0.2(B/L)) \)

\( B \) is the excavation width, and \( L \) is excavation length

If \( D < 0.7B \), then \( T = D \); if \( D \geq 0.7B \), \( T = 0.7B \)

2. Based on the FS calculated above and the wall stiffness, the initial maximum lateral wall movements, \( \delta_{w(max)}^* \) is estimated from Figure 2.2.5 [Clough et al., 1989].

3. Adjustments accounting for strut stiffness and spacing \( (\alpha_s) \), depth to firm layer \( (\alpha_D) \), and excavation width \( (\alpha_b) \) (see Figure 2.2.4) are incorporated for final estimate of maximum wall deformation,

\[
\delta_{w(max)} = (\alpha_s)(\alpha_D)(\alpha_b)\delta_{w(max)}^*
\]

4. The maximum settlement and maximum lateral wall movement are assumed to be equal, \( \delta_{v(max)} = \delta_{w(max)} \).

5. The settlement profile is obtained using the recommended distribution of the surface settlement shown in Figure 2.2.6\(^{10}\).

\(^{10}\) Smith (1987) also incorporated two additional features within MOVEX: (1) accounting for the initial cantilever lateral wall movement and (2) considering the soil strength anisotropy.
In summary, the empirical correlation presented above estimates ground movements on the basis of soil type, workmanship, system stiffness, and the factor of safety against basal heave (based on soil strength). These variables represent a small subset of the possible factors affecting excavation performance summarized in Table 2.1.1 and Figure 2.1.1. Obtaining similar relationships for other factors using this empirical approach is rather difficult since capturing the isolated effects of the various factors requires significant number of well documented and well controlled case studies. Accessing and analyzing such a large number of case studies are difficult, if not impossible, therefore, many have resorted to performing complementing numerical analyses which are capable of modeling actual constructions. Analysis of this type is described in the next section.

2.2.1.2 Numerical Analysis

The finite element, finite difference, and boundary element methods are some of the numerical techniques currently used to analyze excavations. Regardless of the numerical technique used, all analyses have the same goal: estimate the behavior of the excavation through models which meet all the theoretical requirements and boundary conditions that properly reflect the various components of an excavation. Once the numerical model is established, the relationships between the various input parameters and the performance of the excavation, \( \frac{\partial f(x_i)}{\partial x_i} \), Figure 1.1.1) can be determined by varying the factor of interest and monitoring the resulting predicted behavior. This thesis focuses on using the finite element method.

The first finite element analyses for modeling excavations were performed in the early 1970's. Morgenstern and Eisenstein [1970] examined the effects of boundary conditions and changes in lateral pressure
distributions within an elastic medium. Wong [1971] examined the performance of braced excavations; and Tsui [1974] studied the behavior of tie-back walls. With advancements in both computer hardware and software, the use of finite element analysis to model excavations is now much more sophisticated and more widely used. Despite the advancements in computer technology and easier access to the finite element software, the predictive capabilities of finite element analyses are still rather limited. This inadequacy is not due to the method itself, but stems from the inability of developing and using constitutive models to describe soil behavior and actual field conditions.

Most finite element analyses of excavations use very simple constitutive models to describe the soil behavior. Linearly elastic-perfect plastic, hyperbolic, and Modified Cam Clay are three of the most commonly used models to simulate soil behavior. Unfortunately, the behavior of most soils are much more complex and always non-linear. In most cases, the use of these simple models tend to exaggerate the deformations and the zone of influence unless the soil model accounts for increased soil stiffness at very small strains [Puller, 1996]. The failure to properly capture the soil behaviors in the finite element analysis naturally leads to questionable predictions. For this reason, the incorporation of the MIT-E3 model in the ABAQUS program, described in Section 2.3, is a rather significant and promising advancement in analyzing excavation performance by numerical methods.

2.2.2 Design of Supporting Structures

The excavation support structure for deep excavations consists of a supporting wall and wall supports. Sheetpiles and concrete diaphragm walls are two of the more commonly used support wall types for excavations, while
prestressed cross-lot bracing or tieback anchors are typical high stiffness support systems. Structural design focuses on ensuring the support system can withstand the projected loads and control the movements to within specified allowable limits. Similar to predictions of ground movements, there are no standard recommendations for the estimating the design lateral earth pressures to be used in design. The most common design practice are briefly outlined in the following sections.

2.2.2.1 Excavation Support Wall

The design of the supporting wall consists of four general aspects: (a) selecting appropriate wall type/stiffness; (b) adequate embedment depth to assure stability; (c) adequate strength to withstand projected loads; and (d) adequate embedment depth to control flow into the excavation. The first issue, wall stiffness, is closely related to the prediction of excavation-induced deformations. It has been observed that, in soft clay with low factor of safety against basal heave, stiffer walls are effective in reducing ground movements (see Figure 2.2.5). Therefore, the wall stiffness is generally an integral part of most recent design charts for estimating ground movements.

The problem of excavation stability are generally addressed by limit equilibrium methods. Many studies have suggested that increased wall embedment depth can improve the excavation stability by avoiding rotational failure [ASCE, 1996]11. For walls not keyed into underlying rigid layer, the "free earth method"12 is one of the more commonly used method for analyzing the stability of a supported wall. This method assumes that, at failure, the completely rigid wall will rotate about the bracing point (see

---

11 Note that certain deep-seated failures are independent from the wall embedment depth.
12 The fixed earth method is applicable for walls with free top and fixed bottom.
Figure 2.2.8) with passive pressure on the excavation side and active pressure on the retained side. The minimum embedment depth, $t$, is determined by solving for $t$ such that the moment about the bracing point is zero. The required force for the support is determined by satisfying force equilibrium, $\Sigma F_x = 0$. The required strength of the wall, in terms of bending moment, can be determined by calculating the moment distribution in the wall given this assumed loading conditions as shown in Figure 2.2.8 and using method suggested by Blum [see Figure 4-61 in Xanthakos, 1994].

Occasionally, the excavation support wall is also used as a cutoff wall for controlling ground water inflow into the excavation. For excavations in clay, this is generally not a critical issue unless a long construction period is anticipated. Nevertheless, the required length of the wall is constrained by the anticipated flow conditions.

2.2.2.2 Excavation Wall Support

The design of the wall supports involve three issues: (1) stiffness and spacing of the supports; (2) the support forces; and (3) strength and reliability of the supports. The design of the wall support is important since the support wall, support stiffness, and support spacing are the three components that define the support system stiffness which directly affects to the pre-failure deformations (Figure 2.2.5). In the design of wall supports, some consideration is given to the anticipated wall deformations; but the selection of support stiffness and spacing generally relies on practical issues such as appropriate support type, minimum spacing to accommodate construction activities, and possible legal issues [Xanthakos, 1991 and 1994]. Unfortunately,
there are no well accepted design recommendations addressing the proper and ideal wall stiffness and spacing\textsuperscript{13}.

However, there are design methods that recommend anticipated forces on the struts and proper preloading schemes for controlling soil deformations or estimating projected load. The most widely used method for estimating earth pressure for the design of lateral wall supports is the empirical apparent earth pressure diagrams proposed by Peck [1969]. These diagrams were based on actual field measurements of strut loads for excavations supported by sheet piles and soldier piles. Since these pressure distributions were not based on direct measurements of lateral earth pressures, but based on measured strut loads, these distributions are referred to as the apparent lateral earth pressure diagrams. Figure 2.2.9 outlines Peck's recommended earth pressure diagrams. Table 2.2.1 summarizes some of the recommended methods for selecting prestress loads in tieback anchors [Xanthakos, 1991]. It is unclear which method yields the best performance; nevertheless, Peck's charts are the most often cited and the most well-known.

The final issue that the design of wall supports must address is the strength and reliability of the proposed wall supports. For cross-lot bracing, the design of the struts relies on the typical design specifications for steel sections. The design load is based on the estimated lateral earth pressure. For tiebacks in soil, a more site-specific criteria are imposed which usually involve actual field anchor performance tests to ensure the strength of each tieback anchor. In addition to the tieback performance, anchored walls must consider the general stability of the ground-anchored wall system as illustrated in Figure 2.2.10. A number of methods based on limiting

\textsuperscript{13} Clough and O'Rourke's design chart (Figure 2.2.5) does not consider the support stiffness.
equilibrium are available for evaluating this general stability [Xanthakos, 1991].

2.3 Previous Finite Element Analyses Performed at MIT

This research is an extension of Hashash's [1992] study of excavation behavior by incorporating the MIT-E3 soil model in a commercially available general purpose finite element program called ABAQUS. The MIT-E3 soil model, developed by Whittle [1987] is an advanced constitutive model which captures the nonlinear and inelastic behavior of normally consolidated to lightly overconsolidated clay (OCR < 8). Hashash [1992] integrated MIT-E3 within the ABAQUS program in order to study the performance of deep excavations in clay. The ABAQUS finite element program, consists of a large material library, is capable of performing analyses of coupled flow and deformation and can accommodate user-defined material models. Through the use of the ABAQUS program, Whittle and Hashash [1994] compared the predicted behavior of excavations using the MIT-E3, Modified Cam Clay (MCC), and linear elastic soil models. These comparisons show that MIT-E3 is capable of producing accurate and realistic prediction of stress and deformation patterns that enable a better understanding of the excavation behavior. One goal of this thesis is to extend and expand this basic understanding of excavation behavior through additional parametric analyses presented in Chapter 3. Section 2.3.1 provides a brief description of the MIT-E3 soil model and Section 2.3.2 describes the implementation of this soil model in the ABAQUS program.
2.3.1 Description of the MIT-E3 Soil Model

The MIT-E3 model is a relatively complex, generalized soil model which captures three important characteristics of $K_0$-normally consolidated and lightly overconsolidated clays (OCR < 8): (1) small-strain non-linearity; (2) anisotropic stress-strain-strength; and (3) hysteretic and inelastic behavior due to cyclic loading. The model requires 15 material constants (see Table 3.1.1) and a series of variables describing i) the effective stress tensor ($\sigma', S$); ii) the size and orientation of the bounding surface ($\alpha', b$); iii) the effective stresses at the most recent reversal state ($\sigma'_{rev}, S_{rev}$); iv) the strain accumulated since the last reversal state ($\Delta L_E, \Delta L_E$); v) the size of the bounding surface at the last reversal state, $\alpha'_{rev}$; and vi) the size of the load surface at first yield, $\alpha'_{oi}$. Figure 2.3.1 illustrate the physical meaning of these state-dependent variables.

This soil model has been validated at the element level through extensive comparisons with various laboratory test data for Boston Blue Clay (BBC) using the 15 material constants selected especially for resedimented BBC (Whittle et al., 1994). Figure 2.3.2 shows the comparison of measured stress path obtained from plane strain active (PSA) and plane strain passive (PSP) shear tests performed by Ladd et al. [1971] and Directional Shear Cell (DSC) results obtained by Seah [1990] versus MIT-E3 model predictions. Also shown in Figure 2.3.2 are predictions made by the MCC model. As shown in this figure, the predicted stress paths by MIT-E3 model achieved better agreement with the measured data than the MCC model. Figure 2.3.3 shows the corresponding predicted and measured shear stress-strain response of $K_0$-normally consolidated BBC. The predictions by MIT-E3 are in good agreement with the measured data while the MCC model has severe limitations in predicting the anisotropic stress-strain-strength behavior of BBC.
Detailed descriptions of the formulation and coding of the MIT-E3 model are presented in Whittle [1987], Whittle and Kavvadas [1994], and Whittle et al. [1994].

2.3.2 Implementation of the MIT-E3 model in ABAQUS

The ABAQUS program allows the user to add additional mechanical constitutive models to the material library through a subroutine called UMAT. The UMAT subroutine performs two tasks: (1) it calculates the material Jacobian matrix, [K], and (2) it updates the stresses and state-dependent variables at the end of each increment. Hashash [1992] developed a simple algorithm to compute and integrate MIT-E3 in the ABAQUS as a UMAT subroutine: (1) use simple Euler type integrating with substepping to integrate the constitutive equations explicitly, and (2) estimate numerically the consistent Jacobian from the explicit stress calculations. Hashash performed extensive numerical experiments to verify that this scheme is capable of producing accurate and stable calculations of the MIT-E3 model in the ABAQUS program with good convergence properties.

2.4 Description of Current Work

Previous FE analyses of excavations performed by Hashash [1992] assumed that: 1) the excavation length is much larger than the other characteristic dimensions (depth of excavation and excavations width), hence plane strain assumptions can be used in the analysis; 2) the excavations are supported by perfectly rigid struts or cross-lot bracing; and 3) concrete diaphragm walls are used to support the excavation.

In practice the excavation length is usually comparable to other dimensions [e.g. basements of buildings, cf. Whittle et al., 1993]; even long
underground transportation links are often excavated in short sections. It is currently impractical to perform 3-D non-linear analyses of excavations, however, the 3-D effects of the excavation geometry can be inferred by comparing results for plane strain and axisymmetric conditions. Section 2.4.2 describes the implementation of subroutines for performing undrained axisymmetric braced excavations in clay.

Section 2.4.3 focuses on the modeling of the two components of the excavation support system. Modeling of support walls, other than the concrete diaphragm wall is presented in Section 2.4.3.1. The simulation of pre-stressed and passive struts are described in Section 2.4.3.2. Tie-back anchors are now widely used for bracing, even for excavations in relatively deep layers of soft clay, and are essential for wide excavations [e.g. South Boston; Whelan, 1995]. Section 2.4.3.3 outlines the numerical simulation of tie-back anchors used in conjunction with the ABAQUS finite element program.

Hashash performed most of his analyses using Version 4.9 of ABAQUS; since 1992, the ABAQUS program has made few major revisions and the analyses performed in this thesis uses ABAQUS Version 5.4. In response to the changes in the ABAQUS program, modifications had to be made in the excavation procedure. These modifications are described in Section 2.4.1.
2.4.1 Modifications of the Excavation Procedure in ABAQUS

ABAQUS Version 5.4 contains a number of changes compared to Version 4.9 used previously. Consequently, a few modifications and additions are required in response to the version update. These changes can be classified into two general categories: (a) fundamental changes within the ABAQUS input file in terms of modeling the excavation procedure; and (b) modifications in two existing user subroutines\(^{14}\) to ensure compatibility with the updated version. These modifications are summarized in the following paragraphs\(^{15}\):

\[\textit{a. Fundamental Changes in the Excavation Procedure in ABAQUS}\]

One of the major changes in ABAQUS\(^{16}\) with respect to soil modeling is the use of total versus excess pore pressure formulation. In ABAQUS Version 4.9, pore pressure solutions are expressed in terms of "excess pore pressure", which is defined as the pore pressure in excess of the hydrostatic pore pressure; and the effective stresses are calculated based on the given buoyant unit weight, the "excess pore pressure", and the depth of the element. One drawback with this formulation is that the gravity effects of the water are not included directly in the element. When modeling excavations, the gravity effects of the water are very important since the excavation involves the removal of both the pore water and the soil skeleton. Under the original excess pore pressure formulation, the soil skeleton and the pore fluid are treated as two separate entities; thus when soil elements are removed, only the corresponding soil particles are removed, the pore fluid is left behind.

---

\(^{14}\) The two existing subroutines are UMAT for defining user material constitutive model and SIGINI for defining the initial geostatic stress state.

\(^{15}\) Sample ABAQUS input files are shown in Appendix A.

\(^{16}\) For versions including and after V5.1.
To account for the water removal, applications of additional distributed loads equivalent to the pore pressure need to be applied. Therefore, under version 4.9, the simulation of an excavation step consists of the removal of the appropriate soil elements as well as the application of additional distributed loads to account for the water removal [see Figure 3.5 in Hashash, 1992].

Under the total pore pressure formulation, available in Version 5.4, the soil particles and the pore fluid can be treated as one entity. This feature simplifies the excavation procedure within the program since element removal accounts for the removal of the soil particles as well as the pore fluid.

The second major change within ABAQUS is the non-linear solution controls. In Version 4.9, convergence for stress-strain analysis is defined by a single tolerance measure on the force equilibrium. Essentially, convergence is achieved when the force residuals are below this pre-specified tolerance. For simpler material models, stringent convergence criteria can be achieved rather efficiently. In the case of the MIT-E3 soil model, an explicit integration algorithm is used to approximate the Jacobian stiffness matrix. This technique is unable to achieve very tight force tolerances; however, displacements tolerances provide a more reliable indicator of convergence characteristics for excavation problems [Hashash, 1992]. Consequently, convergence is based on the maximum displacement residual in two successive iterations of the global Newton scheme. In version 4.9, this displacement-based convergence criterion was imposed manually.

---

17 This force equilibrium tolerance is referred to as PTOL in ABAQUS, which is the ratio between the force residual and the average nodal force.
requiring the program to be restarted after each step resulting in a slow, time-consuming process. In version 5.4, non-linear convergence criteria in terms of both the force and displacement tolerances\textsuperscript{18} can be specified. The time stepping can therefore be automated with experience.

\textit{b. Modifications in the User Subroutines UMAT and SIGINI}

There are four changes with respect to the user subroutines implemented in ABAQUS. The first three are minor additions in the interface cards and the declaration of the computer type; the fourth addition involves the use of a new user subroutine or input statement for the purpose of ensuring proper initialization of variables.

The first change involves additions of variables to the argument list for user material subroutine UMAT in ABAQUS version 5.4. These additional variables can be used to control time incrementation within the UMAT subroutine. Since there is no need to incorporate time incrementation within UMAT at this time, the only required modification is to add these variables to the interface card to assure compatibility without any other modifications to the subroutine itself.

The second change consists of similar additions of variables to the argument list but for user subroutine SIGINI, which is used to define the initial geostatic stresses; likewise, there are no changed in the remaining codes within subroutine SIGINI.

The third change involves declaration of double precision machine or single precision machine. To avoid any confusion or incorrect specification, version 5.4 contains an new command,

\textsuperscript{18} Version 5.4 includes an additional option of using the ration of largest displacement correction to the largest incremental solution as a convergence criterion.
INCLUDE 'ABA_PARAM.INC', to be added at the beginning of the user subroutines. This command will invoke procedures which automatically determine if computer platform is a single or double precision machine.

The fourth change, which is also the most critical change, involves the incorporation of a new subroutine called SDVINI or an additional ABAQUS command in the input file which will ensure proper initialization of the state-dependent variables. The user subroutines used for ABAQUS version 4.9 were written with the presumption that all the variables are initialized to zero. This is true for the FORTRAN compiler used previously; however, on the current workstation\(^\text{19}\) with version 5.4, this assumption no longer holds. Though most variables are unaffected by this change, one variable, the last state-dependent variable, relies on the value being initially set to zero. This last state-dependent variable is used as a flag to indicate that the subroutine UMAT is being called for the first time and consequently initializes all the state-dependent variables to the appropriate initial values. Without the proper initialization of the state-dependent variables, the results obtained from the analysis are essentially useless. In order to resolve this problem, a new user subroutine, SDVINI, or the ABAQUS command "INITIAL CONDITIONS, TYPE = SOLUTION" can be used to ensure that this state-dependent variable is initially zero.

With the modifications described above, the same level of accuracy as solutions obtained by Hashash can be achieved. In fact, the combination of improved computer hardware and convergence control, these solutions can

\(^{19}\text{ All the simulations are performed on a DEC Alpha 3000-300X with UNIX-based operating system (DEC OSF).}\)
be obtained with a significant reduction in computation time. Table 2.4.1 summarizes the computation time required to perform a reference analysis of two model excavation experiments with two soil models (MCC an MIT-E3) using the three workstations currently available. As shown in Table 2.4.1, the identical job can be completed less than 1/10 of the time required by the older workstations.

2.4.2 Implementation of the User Elements for Axisymmetric Undrained Analyses

For typical excavations with limited excavation length, plane strain finite element analyses generally overestimate the measured far field ground movements. Some researchers [Naylor and Pande, 1981; St. John, 1975] have suggested that axisymmetric analyses provide a more realistic geometry for describing these far field deformations. This reasoning stems from the concept that with limited excavation length, kinematic constraints in the far field more closely resemble axisymmetric than plane strain assumptions. Thus, significant research efforts have been carried out to implement undrained axisymmetric analyses for excavations in clay.

2.4.2.1 ABAQUS UEL Subroutine for Undrained Axisymmetric Analyses

Undrained analyses in the clay impose an important constraint of incompressible material behavior that causes numerical difficulties in finite element analyses [Legaspi, 1996]. In general, these problems can be mitigated by ensuring adequate degrees of freedom (DOF) within each finite element. For plane strain analyses, conventional 8-4 mixed elements (Figure 2.4.1a, an element available in the ABAQUS element library) perform satisfactorily. Unfortunately, for axisymmetric analyses, incompressibility introduces six
additional constraints on the displacements and conventional elements in
the ABAQUS element library are no longer adequate.

In order to perform undrained axisymmetric analyses, the solution
proposed by Sloan and Randolph [1982] is adopted. This approach
recommends the use of higher order elements with more nodes per element
(i.e., more DOF's and higher order interpolation functions). Previous studies
[Kavvadas, 1993; Legaspi, 1996; and Geer, 1995] have found that the 15-3
triangular elements (Figure 2.4.1b) with cubic interpolation of strains give
numerically accurate results for undrained axisymmetric analyses.

Unfortunately, this type of high order element is not available in the
element library of ABAQUS program but can be added using user-defined
element subroutine, UEL\textsuperscript{20} available in ABAQUS. This task was first
implemented by Kavvadas [1993], and has been extended by Geer [1995] to
include the MIT-E3 model. Further modifications of the subroutines have
been carried out in this research for the purpose of expanding the capability of
simulating undrained axisymmetric excavations. The next section, Section
2.4.2.2, summarizes the three major modifications to the UEL subroutine.

2.4.2.2 Modifications to the Original UEL Subroutine

There are two major limitations in the Original UEL subroutine which
prevents the simulation of undrained axisymmetric excavations: (a) element
removal is virtually impossible; and (b) the entire mesh is required to have
the same element type. The following three modifications were made in
order to accommodate undrained axisymmetric excavations:

1. **Total Residual Vector instead of Incremental Residual Vector**

\textsuperscript{20} The UEL subroutine requires a complete assembly of the element stiffness matrix and the
load vectors.
The output requirement for the UEL subroutine within ABAQUS is to update the tangent stiffness matrix (Jacobian, [K]) and the residual vector (right-hand-side, [R]) in Equation 2.4.1 for each element subject to prescribed boundary conditions and displacement increments.

\[
\begin{bmatrix}
K & -L \\
-L^T & -\Delta t \cdot H
\end{bmatrix}
\begin{bmatrix}
dU \\
dP
\end{bmatrix} = 
\begin{bmatrix}
R_u \\
R_p
\end{bmatrix}
\] (2.4.1)

In the original formulation by Kavvadas [1993], the right hand load vector, [R], consists of the incremental rather than the total residual vector (see Equation 2.4.2). However, in excavation analyses, element removal requires the release of corresponding total nodal forces values which are not readily available at a given step in the original formulation. Therefore, the original code has been modified such that the right-hand-side consists of total residuals (see Equation 2.4.3).

\[
\begin{bmatrix}
K & -L \\
-L^T & -\Delta t \cdot H
\end{bmatrix}
\begin{bmatrix}
dU \\
dP
\end{bmatrix} = 
\begin{bmatrix}
\left(R^{i+\Delta t} - R^i\right) - \left\{ B^T \cdot \left(\sigma^{(i-1)'} - \sigma^i\right) dV - L \cdot \left(P_{(s)}^{(i-1)} - P_{(s)}^i\right) \right\} \\
\Delta t \cdot Q^{i+\Delta t} + \Delta t \left\{ H^{(i-1)} \cdot P_{(s)}^{(i-1)} + L^T \cdot V^{(i-1)} \right\}
\end{bmatrix}
\] (2.4.2)

\[
\begin{bmatrix}
K & -L \\
-L^T & -\Delta t \cdot H
\end{bmatrix}
\begin{bmatrix}
dU \\
dP
\end{bmatrix} = 
\begin{bmatrix}
\left(R^{i+\Delta t} - R^i\right) - \left\{ B^T \cdot \left(\sigma^{(i-1)'}\right) dV - L \cdot \left(P_{(s)}^{(i-1)} - P_{(s)}^i\right) \right\} \\
\Delta t \cdot Q^{i+\Delta t} + \Delta t \left\{ H^{(i-1)} \cdot P_{(s)}^{(i-1)} + L^T \cdot V^{(i-1)} \right\}
\end{bmatrix}
\] (2.4.3)

2. Additional Load Type U14 for Identifying the Removed Elements

Elements are numerically removed (i.e. excavated) from a mesh by omitting the contributions of these deactivated elements to the stiffness matrix, [K] and the residual vector, [R], from the global

\[\text{See Kavvadas (1993) for a detailed derivation of these equations and explanation of each components.}\]

\[\text{For FE analyses that do not involve element removal, the total nodal forces are not critical. Correct stress-strain calculations can be obtained with the original formulation; and the total nodal forces can be calculated after the completion of the analysis by adding the incremental nodal forces reported at each step.}\]
assembly of equations. It was determined from simple numerical tests that ABAQUS includes all user elements in its global assembly of equations regardless of whether or not the user element is activated. Therefore, to remove an user element using the ABAQUS program, the corresponding stiffness matrix, \([K]\), and the residual vector, \([R]\), must be set to zero. To accomplish this, an additional load type, U14, was added to the UEL code for the purpose of flagging removed elements. This load type, U14, is defined as a typical distributed load using the following ABAQUS command in the input file [*.inp file]:

```
*DLOAD

[Element Number], U14, [Magnitude]
```

Removed elements are identified by having an active load type, U14; and consequently, the corresponding element stiffness matrix, \([K]\), and the residual vector, \([R]\), are set to zero thus having zero contribution to the global assembly of equations.

3. Additional Material Type #5 in order to use Solid and Mixed (soil) Elements in the same Mesh

The original UEL subroutine did not allow different type of elements (e.g. solid and soil) within the same mesh. This "feature" in the original UEL poses a significant problem for the analysis of braced excavation where both the soil (mixed elements) and support structures (represented by solid elements) are present. This restriction of elements in the mesh stems from the variable storage format specified by ABAQUS. Essentially each node has a pre-specified active degrees of freedom (DOF), and these DOF's must be active at all times. Consequently, solid elements cannot be placed adjacent to mixed elements due to incompatibilities in the array representing the active
DOF's (mixed elements have one additional DOF associated with the pore pressure compared to solid elements). This problem has been resolved by adding another material type, #5, in the UEL subroutine which defines the constitutive response of mixed elements to respond as a solid. This material type essentially defines a mixed-element which behaves like an isotropic elastic solid, therefore allowing this "elastic solid" to be placed in the same mesh with mixed (soil) elements.

The UEL code with these three modifications has been tested extensively both at the element level (through the comparison of triaxial test results obtained from using ABAQUS elements versus UEL elements) and at the mesh level (through comparison of plane strain excavation analyses using ABAQUS 8-4 elements versus plane strain UEL 15-3 elements). All the tests performed suggest that this modified UEL is capable of performing the correct calculations in terms of element removal and the stress-strain behavior of the solid wall within the soil mass under axisymmetric conditions.

2.4.2.3 Comparison of Axisymmetric and Plane Strain Excavations

Figure 2.4.2 shows the predicted surface settlements and wall deflections of five model undrained excavations in normally consolidated Boston Blue Clay\textsuperscript{23} at an excavation depth of $H = 12.5$-m. The first model excavation is 80-m wide with depth to bedrock of 50-m supported by 0.9-m thick, 40-m long concrete diaphragm wall ($E = 2.26\times10^4$ MPa and $\nu = 0.15$) with

\textsuperscript{23} See Section 2.3 and Chapter 3 for more detailed description of the behavior of excavations in normally consolidated BBC.
rigid struts spaced at every 2.5-m. The remaining four excavations are assumed axisymmetric with a diameter of 80-m and depth to bedrock of 50-m. These four excavations are also supported by 40-m long walls with rigid struts spaced at every 2.5-m. However, the four walls have Young's moduli that are equal to, 1/2, 1/10, and 1/20 of the modulus for a 0.9-m thick concrete diaphragm wall. As shown in this figure, changes in the wall stiffness generally influence the magnitudes of the displacement within the trough portion of the surface settlement (x < 70-m). At 70-m from the support wall, the settlements from the axisymmetric cases are independent of the wall stiffness and are approximately 0.3-cm. The plane strain case, on the other hand, generates a settlement of approximately 0.8-cm at 70-m from the wall. This simple comparison of axisymmetric versus plane strain analyses suggests that a reduction in the excavation length can reduce far field settlements by more than 50%.

More detailed analyses of effect of excavation geometry requires additional modifications to the UEL code. Additional research is needed to determine the appropriate method of modeling the wall in order to match the bending stiffness of a plane strain wall. Nevertheless, the results shown in Figure 2.4.2 suggests that far-field settlements are independent from the wall stiffness and illustrates the improvements of axisymmetric analysis on far-field settlement calculations.

2.4.3 Modeling of Support Systems

The lateral earth support system generally consists of two components, the wall itself and the bracing supports. This section describes how these components are modeled in the analyses presented in Chapters 3 and 4. In order to simplify the analyses, issues relating to wall and tieback installation,
and the imperfect contact between cross-lot bracing and the support walls are not addressed in the current study. Section 2.4.3.1 describes the technique used to account for the wall stiffness in the support walls. Sections 2.4.3.2 and 2.4.3.3 present the procedure used for modeling bracing struts and tiebacks.

2.4.3.1 Support Wall Stiffness

Soldier piles and lagging, sheet piles, bored piles, and concrete diaphragm walls are some of the excavation support walls used in practice. The key engineering properties of the walls are their axial stiffness, bending stiffness, and strength. In most cases, the walls have a well defined linear elastic stress-strain response prior to yielding. For this reason, the walls are modeled as two columns of elastic solid elements using plane strain 8-noded isoparametric solid elements with elastic material properties. The size of the elements and the corresponding elastic properties are obtained by matching the axial and bending stiffnesses with the actual section (sheetpile) following the recommendation of Day and Potts' [1993].

According to Day and Potts, the bending behavior of the wall can be modeled using 2-D plane strain solid elements by selecting an equivalent wall thickness, $t_{fe}$, elastic modulus, $E_{fe}$, and a Poisson's ratio of between 0.1 to 0.15. The equivalent wall thickness, $t_{fe}$, and elastic modulus, $E_{fe}$, are calculated by matching the bending, $E_sI_s$, and axial stiffnesses, $E_sA_s$, of the actual wall section using the following equations for each unit length$^{24}$:

$$E_sI_s = E_{fe} \frac{t_{fe}^3}{12} \quad (2.4.4) \quad \text{(matching bending stiffness)}$$

$$E_sA_s = E_{fe}t_{fe} \quad (2.4.5) \quad \text{(matching axial stiffness)}$$

Rearranging these two equations:

$^{24}$ Moment of inertia, I, for a rectangular section around the centroidal axes is $I = bh^3/12$ (note: $b = 1$ unit).
\[ t_{fe} = \sqrt{\frac{12I_s}{A_s}} \quad (2.4.6) \]
\[ E_{fe} = \frac{E_s A_s}{t_{fe}} \quad (2.4.7) \]

2.4.3.2 Modeling of Passive and Pre-stressed Struts

The modeling of bracing struts are very similar to the modeling of internal floor slabs described by Hashash [1992]. The finite, laterally spaced struts are modeled as equivalent distributed elastic spring elements with no moment connection to the wall (see Figure 2.4.3). The axial stiffness for these elastic springs are obtained by the following relationship:

\[ k_s = \frac{k'}{L} = \frac{E_s A_s}{(B/2)S} \quad (2.4.8) \]

where \( k' \) is the spring constant [force/length]; \( L \) is the unit length of the wall [length]; \( E_s \) is the Young's modulus of the steel [force/length²]; \( A_s \) is the cross sectional area of the steel strut [length²]; \( B/2 \) is the half-length of the cross-lot strut = half-width of the excavation [length]; \( S \) is the average lateral spacing of the strut [length].

Two types of bracing are considered in this thesis: passive supports and pre-stressed struts. Passive supports refer to struts that are fixed in place at the time of installation without any pre-specified loads. Any subsequent changes in the strut forces are in direct response to the movements of the wall. Figure 2.4.4 illustrates the modeling of passive struts.

Pre-stressed struts, on the other hand, have a pre-defined level of load at the time of installation. In practice, there are two reasons for using pre-stress struts: (1) correct pre-stress schemes can reduce wall movements; and (2) pre-stressing eliminates gaps between the wall and the strut, thus resulting
in strut deformations that are closer to their intended performance. Figure 2.4.5 illustrates the modeling of pre-stressed struts.

2.4.3.3 Modeling of Tiebacks

Numerous research studies have focused on the behavior of tieback anchors using finite element analysis [Clough and Tsui, 1974; Prieto-Portar, 1979; Simpson et al., 1979; Desai et al., 1986; and Fernandes and Falcão, 1988]. The numerical models used in these studies vary in complexity depending on the goal of the research. In general, one dimensional spring or bar elements are used to simulate the axial load-deformation behavior within the free length. A general model of the anchor fixed length would be able to replicate the pull-out capacity and installation disturbance, while interface elements would model the stick-slip behavior. However, for loading well below the pull-out capacity of the tieback, deformations of the bonded length are assumed to be small compared to the elongation of the steel tendon within the free length. Hence, the current analyses incorporate the axial elongation of the bonded length and do not model the interface properties or installation effects on the surrounding soil.

For the South Boston ISS-4 Section (see Section 4.2), the design load of the tiebacks was set at 50% of the failure load [Whelan, 1995]. Proof tests, which consist of loading the anchor to 133% to 150% of the design load, were performed on all tieback anchors in clay. Similar tests were also performed on anchors in rock. The acceptance criteria for the proof tests ensure that minimal slippage occurs in the bonded zone. Every tieback anchored in clay or rock must pass this proof test: If the tieback fails the proof test, it is regrouted until the acceptance criteria are met. This procedure guarantees the performance of the bonded zone; therefore, the assumption of no slippage
between the anchor fixed length and surrounding soil is valid for the CA/T South Boston site.

Figure 2.4.6 shows the numerical model used to simulate the tiebacks anchored in bedrock. This model is very similar to the numerical model used for prestressed cross-lot bracing (see Figure 2.4.5). The only differences between the two are (1) rock anchors are inclined, and (2) the initial pre-stress is a tensile force rather than compressive load. The equivalent stiffness for the rock anchor can be calculated using equation 2.4.8\(^{25}\) or obtained directly from the measured stiffness.

The model for tiebacks anchored in clay is more complex than rock anchors. The entire tieback, fixed length and free length, is modeled using a series of 4 springs: one spring representing the free length and 3 springs representing the fixed length. Figure 2.4.7 illustrates the model for tie-backs in clay, including the application of the lock-off load and the stress-strain behavior after lock-off assuming no installation effects. In this model, the displacements in the tieback are described by the elongation of the steel tendon (spring \(a\)) and the elastic axial stiffness of the bonded length (springs \(b_1, b_2,\) and \(b_3\)^{26}) which is free to displace with the surrounding soil. The application of lock-off load is modeled by applying a set of equal and opposite force at the location of the tieback anchorage on the support wall and at the head of the bonded length. The magnitude and the direction of the force is equal to the actual lock-off load per unit length. After the application of the lock-off load, spring \(a\), representing the free length, is introduced to constrain the movements between the tieback anchorage and the head of the bonded length. This method is very similar to method used by Tsui and Clough

\(^{25}\) Instead of (B/2), the actual free length of the tieback should be used.
\(^{26}\) Three springs are used to model the fixed length in order to account for the reduction in axial stress along the bonded length.
the only difference is that spring elements, instead of bar elements, are used. The implementation of the prestressing operation in ABAQUS generally follows the same procedure as described above. Figures 2.4.8 and 2.4.9 describe the procedure used to implement this model in ABAQUS. As shown in Figures 2.4.8 and 2.4.9, additional imaginary nodes are used to track the relative displacements of the tieback anchorage on the wall and the head of the bonded length.

Table 2.4.2 summarizes the typical properties and dimensions of tiebacks in clay as well as clay anchors used at the South Boston ISS-4 site. Figure 2.4.10 illustrates how the equivalent stiffnesses for the four springs can be obtained using the field-measured overall stiffness.

The effect of this prestressing process is illustrated using results from eight model undrained excavations within normally consolidated BBC excavated to a final depth of \( H = 7.5 \text{-m} \). Figure 2.4.11 describes the eight analyses: (a) one base case with perfect struts\(^{27}\) located at 2.5-m and 5.0-m; and (b) seven excavations with varying level of prestress in the tieback at a depth of 2.5-m inclined at 22° and a perfect strut at 5.0-m. The properties of the tiebacks are summarized in Figure 2.4.11. The seven prestress levels selected are 49 kN/m, 98 kN/m, 147 kN/m, 196 kN/m, 245 kN/m, 392 kN/m, and 490 kN/m\(^{28}\).

Figures 2.4.12 and 2.4.13 show the wall deflections and surface settlements of these eight cases at excavation depths of \( H = 5.0 \text{-m} \) and \( H = 7.5 \text{-m} \)\(^{29}\), respectively. Increases in the lock-off load result in increased

\(^{27}\) Perfect strut is defined as no further movement at the point of strut location after installation.

\(^{28}\) In the base case, the perfect strut located at 2.5-m reported a horizontal load of 220 kN/m at the time of installation (no vertical force component).

\(^{29}\) These eight excavations have identical initial cantilever movements after the unsupported 2.5-m of excavation.
movements into the retained soil at the top of the wall. The wall deflections below the excavated grade reflect some influence of the vertical component of lock-off load. Nevertheless, 20-m below the excavation grade, the support type has negligible effect on the lateral movement. Similar trends are also observed in the surface settlement. Magnitudes of the trough portion vary inversely with the lock-off load. Beyond 40-m from the support wall, the magnitude of the lock-off load has negligible effects on the surface settlement.

Figure 2.4.14 and 2.4.15 summarize the total horizontal and vertical movements at the tieback lock-off point (at depth of 2.5-m) and at the head of the fixed length, as functions of the tieback prestress and excavation depth, H. As the prestress load increases, the lock-off point moves into the retained soil, however the fixed length also moves toward the wall (Figure 2.4.14)\(^{30}\). In terms of vertical movements (Figure 2.4.15), the lock-off load has negligible effects on the vertical movements of the tieback anchorage; however, an increased upward movement in the head of the bonded zone was observed with increases in lock-off load.

2.5 Application of Finite Element Capabilities

The basic modeling tools described in this chapter is applied in the subsequent chapters. The use of strut and sheetpile walls are included in the parametric studies presented in Chapter 3 as well as the South Cove and MBTA Transitway case studies analyzed in Sections 4.1 and 4.3. The use of tieback anchored in rock and clay is illustrated in the CA/T South Boston case study presented in Section 4.2.

\(^{30}\) Movements of the fixed anchor in this example become unrealistically large for prestress loads greater than about 200kN/m.
<table>
<thead>
<tr>
<th>Site Conditions</th>
<th>Design Parameters</th>
<th>Construction Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>2. Ground water</td>
<td>2. Stiffness of supports</td>
<td>2. Duration of construction</td>
</tr>
<tr>
<td>3. Existing structures and utilities</td>
<td>3. Support Spacing</td>
<td></td>
</tr>
<tr>
<td>4. Transient surcharge loads during and after</td>
<td>4. Depth and width of</td>
<td></td>
</tr>
<tr>
<td>construction</td>
<td>excavation</td>
<td></td>
</tr>
</tbody>
</table>

Table 2.1.1 - Typical Factors Cited which Influence Excavation Performance [modified based on Mana and Clough, 1981]

<table>
<thead>
<tr>
<th>Reference</th>
<th>Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kapp</td>
<td>Percentage of allowable tie-rod load (20-60%)</td>
</tr>
<tr>
<td>Mansur and Alizadeh</td>
<td>At-rest Pressures</td>
</tr>
<tr>
<td>Rizzo, et al.</td>
<td>Active to at-rest</td>
</tr>
<tr>
<td>Shannon and Strazer</td>
<td>50% anchor yield load</td>
</tr>
<tr>
<td>Clough, et al. (1974)</td>
<td>Terzaghi-Peck rules (0.4γH)</td>
</tr>
<tr>
<td>Liu and Dugan</td>
<td>15 X height of wall (in psf)</td>
</tr>
<tr>
<td>Hanna and Matallana</td>
<td>Pressures halfway between active and at-rest</td>
</tr>
<tr>
<td>Oosterbaan and Gifford</td>
<td>Active Pressures</td>
</tr>
<tr>
<td>Larson, et al.</td>
<td>Pressure between active and at-rest</td>
</tr>
</tbody>
</table>

Table 2.2.1 - Summary of Methods Used to Estimate Prestress Load on Tieback Anchors [from Xanthakos, 1991, p.551]
## Table 2.4.1 - Comparison of Computation Time for a Model Excavation using Three Different Workstations

<table>
<thead>
<tr>
<th>Model Excavation using MCC</th>
<th>CPU Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>Running on SUN SPARCStation SLC [PUCK]</td>
<td>1:51:08</td>
</tr>
<tr>
<td>Running on DEC Alpha 3000-300X [GCUBE]</td>
<td>0:10:40</td>
</tr>
<tr>
<td><strong>Model Excavation using MIT-E3</strong></td>
<td></td>
</tr>
<tr>
<td>Running on VAX Station 3200 [GEOTECH]</td>
<td>$\geq 20:00:00$</td>
</tr>
<tr>
<td>Running on DEC Alpha 3000-300X [GCUBE]</td>
<td>1:12:37</td>
</tr>
</tbody>
</table>

**Note:**
1. The model undrained excavation consists of a 40-m wide excavation in 120-m deep clay supported by 40-m long 0.9-m thick concrete diaphragm walls with rigid supports spaced at 2.5-m. (812 elements, 2551 nodes)
2. GEOTECH is the machine used by Hashash in his research
3. Both PUCK and GEOTECH are running ABAQUS version 4.9; only GCUBE is running ABAQUS version 5.4.

## Table 2.4.2 - Typical Properties and Dimensions of Tiebacks in Clay

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Free Anchor Length L_t</td>
<td>7 to 26-m [23 to 86-ft]</td>
<td>6 to 15-m [20 to 50-ft] (see note 1)</td>
</tr>
<tr>
<td>Fixed Anchor Length L_f</td>
<td>12-m [40-ft]</td>
<td>10-m [33-ft]</td>
</tr>
<tr>
<td>Inclination Angle, i</td>
<td>20° to 22°</td>
<td>Typically 15° to 30° (see note 2)</td>
</tr>
<tr>
<td>Anchor Lateral Spacing, S</td>
<td>1.8 to 4.4-m [6 to 14.5-ft]</td>
<td>2 to 3-m [6 to 9-ft]</td>
</tr>
<tr>
<td>Tendon Stiffness, E_t</td>
<td>9.8E4 to 1.4E5 MN/m² [14,200 to 20,500 ksi] (see note 3)</td>
<td>1.7E5 to 1.8E5 MN/m² [24,800 to 26,000 ksi] (see note 4)</td>
</tr>
<tr>
<td>Diameter of Drilled Hole, D</td>
<td>14-cm [5.5-in] for tendon consisting of six 0.6&quot; diameter 270ksi strands</td>
<td>12.7 to 15.2-cm [5 to 6 in] for tendon consisting of 7 to 11 0.5&quot; diameter 270ksi strands</td>
</tr>
</tbody>
</table>

**Note:**
1. Usually less than 40-m [130-ft]
2. 45° is used if suitable ground for anchorage is deep
3. The tendon stiffness is the measured overall stiffness
4. This is the typical steel tendon stiffness measured in lab

Page 74
Figure 2.1.1 - Factors Influencing Ground Movements due to Excavations in Soft Ground
I - Sand and Soft to Hard Clay, Avg. Workmanship

II - Very Soft to Soft Clay
   2. Significant Depth of Clay Below Bott. Exc.,
      But \( N_b < N_{cb} \)

III - Very Soft to Soft Clay to a Significant Depth
      Below Exc. Bott. and \( N_b > N_{cb} \)

\[ N_b = \text{Stability No. Using C "Below Base Level" } = \frac{YH}{C_b} \]
\[ N_{cb} = \text{Critical Stab No. for Basal Heave} \]

Figure 2.2.1 - Summary of Observed Settlements behind Sheet Pile and Soldier Pile Walls (Peck, 1969)
Figure 2.2.2 - Relationship between Factor of Safety Against Basal Heave and Normalized Maximum Lateral Wall Movements from Case History Data (Mana and Clough, 1981, p. 746)

Figure 2.2.3 - Relationship Between Maximum Ground Settlement and Maximum Lateral Wall Movement based on Case History Data (Mana and Clough, 1981, p. 765)
Figure 2.2.4 - Recommended adjustment factors for effects of strut stiffness, effect of depth to underlying firm layer, and effect of excavation width (Mana and Clough, 1981)
Figure 2.2.5 - Relation Among Maximum Lateral Wall Movements, System Stiffness, and Factor of Safety Against Basal Heave for Cuts in Plastic Clay [from Terzaghi et al., 1996, p. 462]:
(a) Calculated by Finite-Element Solutions;
(b) Comparison with Field Measurements (after Clough et al., 1989)
Figure 2.2.6 - Envelopes for Normalized Ground Settlement Profiles (Mana and Clough, 1981)
Figure 2.2.7 - Dimensionless Settlement Profiles Recommended for Estimating Settlement Distribution Adjacent to Excavations in Different Soil Types (Clough and O'Rourke, 1990)
Figure 2.2.8 - Active Force and Passive Resistance in Wall assuming Free-Earth Support Conditions (Xanthakos, 1994, p.285)

Figure 2.2.9 - Apparent Earth Pressure Diagrams for Computing Strut Loads in Strutted Excavations (Peck, 1969)
Deep-Seated Failure

Rotational Failure due to Inadequate Penetration

Flexural Failure of the Wall

(a) Modes of Failure for Anchored Wall (ASCE, 1996)

Using Limiting Equilibrium Method assuming circular slip surface

Using Sliding Block Method

(b) Stability Analysis of Tieback Wall (Xanthakos, 1991, p.546)

Figure 2.2.10 - Modes of Failure for Tieback Anchored Walls and Analysis Methods for Evaluating the Stability of the Wall
Figure 2.3.1a - Yield, Failure, and Load Surfaces used in MIT-E3 [Whittle and Kavvadas, 1994]

Figure 2.3.1b - Effects of Stress History on Initial Values of State Variables in MIT-E3 Model [Whittle and Kavvadas, 1994]
Figure 2.3.2 - Evaluation of Stress Paths Predicted by MIT-E3 and MCC Soil Models for Undrained Plane Strain Shear of $K_0$-Normally Consolidated BBC in Directional Shear Cell [Whittle et al., 1994]
Figure 2.3.3 - Evaluation of Shear Stress-Strain and Secant Shear Modulus-Strain Response of K₀-Normally Consolidated BBC in Directional Shear Cell Tests Using MIT-E3 and MCC Soil Models [Whittle et al., 1994]
(a) **8-4 Element**: 8 displacement nodes and 4 pore pressure nodes. [ABAQUS: CPE8P; UEL: Element U7]

(b) **15-3 Element**: 15 displacement nodes and 3 pore pressure nodes. [ABAQUS: none; UEL: Element U19]

Figure 2.4.1 - 2-D Mixed Elements used in Finite Element Analysis of Excavations
Figure 2.4.2 - Wall Deflections and Surface Settlements for Model Plane Strain and Axisymmetric Undrained Excavations at H = 12.5-m
Figure 2.4.3 - Derivation of Equivalent Plane Strain Strut Stiffness
**Step A: Prior to Strut Installation**

DEFINITION:
- Node A is a "virtual" node connected to Node B only
- Node B is a node on the Wall at the location of the strut

**CONSTRAINT 1:**
\[
\delta_v |_{node\ A} = \delta_v |_{node\ B} \\
\delta_h |_{node\ A} = \delta_h |_{node\ B}
\]

1. Remove Constraint 1
2. Impose Constraint 2

(Allow the spring representing the strut is free to deform based on the spring constant $k'$)

**Step B: Strut Installation**

**Step C: After Strut Installation**

**CONSTRAINT 2:**
\[
\delta_v |_{node\ A} = \delta_v |_{node\ B} \\
\delta_h |_{node\ A} = 0
\]

Elevation View

Figure 2.4.4 - Modeling of Passive Strut within Finite Element Analysis
**Step A: Prior to Strut Installation**

DEFINITION:

- Node A is a "virtual" node connected to Node B only
- Node B is a node on the Wall at the location of the strut

**CONTRAINT 1:**

\[
\delta_v \mid_{\text{node } A} = \delta_v \mid_{\text{node } B} \\
\delta_h \mid_{\text{node } A} = \delta_h \mid_{\text{node } B}
\]

**Step B: Strut Installation and Strut Prestress**

1. Remove Constraint 1
2. Impose Preload, \( P \) [F/L]
   (Allow the strut to deform)

**Step C: After Strut Installation**

**CONTRANT 2:**

\[
\delta_v \mid_{\text{node } A} = \delta_v \mid_{\text{node } B} \\
\delta_h \mid_{\text{node } A} = 0
\]

Figure 2.4.5 - Modeling of Pre-stressed Strut within Finite Element Analysis
The tie-back is modeled as a single spring \((A^*-B^*)\) with node \(B^*\) as a node on the wall and node \(A^*\) located at the bedrock, but not tied to the nodes defining the bedrock. The lock-off load for the tie-back is applied at node \(A^*\) and after lock-off, node \(A^*\) is fixed against any displacement.

Figure 2.4.6- Numerical Model of Tieback anchored in Bedrock

\[
(k') = \frac{(AE)}{(L_t S)}
\]
Modeling of Tieback in Finite Element Analysis

Tieback is modeled as a series of four springs: one representing the free length [a] and three representing the bonded length [b₁, b₂, and b₃].

Step A: Prior to Installation

All four Springs are removed from the mesh.

Step B: At Installation

Three springs representing the bonded length are introduced back to the mesh.

Step C: At Lock-off

Concentrated loads equivalent to the Lock-off Load are applied to both the wall and the head of the bonded length.

Step D: After Lock-off

Spring representing the free length is introduced back to the mesh.

Figure 2.4.7-Numerical Modeling of tiebacks and application of lock-off load for tieback anchors in clay
NUMERICAL IMPLEMENTATION OF TIEBACKS: free length

Nodes 101, 80101, and 90101 have the same coordinates.

Nodes 102, 80102, and 90102 have the same coordinates.

Nodes 101 & 102 are part of the mesh representing the soil profile.

Nodes 90101 & 90102 are the two end nodes defining the spring which represents the free length.

Nodes 80101 & 80102 are imaginary nodes not connected to any element.

Displacement constraints imposed on the relative movements of these three sets of nodes:

\[ U_{101} - U_{90101} = U_{80101} \]
\[ U_{102} - U_{90102} = U_{80102} \]

<table>
<thead>
<tr>
<th>NODE SET</th>
<th>DISPLACEMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>101 &amp; 102</td>
<td>total displacement</td>
</tr>
<tr>
<td>80101 &amp; 80102</td>
<td>displacement up to and including lock-off</td>
</tr>
<tr>
<td>90101 &amp; 90102</td>
<td>displacement after lock-off</td>
</tr>
</tbody>
</table>

Figure 2.4.8 - Element Definition for Modeling the Free Length of the Tieback Anchors in Clay using ABAQUS
NUMERICAL IMPLEMENTATION OF TIEBACKS: free length

Steps A & B: Prior to Lock-off

Element freelnength is removed from the mesh
Displacement constraints: \( U_{90101} = U_{90102} = 0 \)

\[ \Rightarrow U_{101} = U_{80101}; \quad U_{102} = U_{80102} \]

Step C: At Lock-off

Element freelnength is removed from the mesh
Displacement constraints: \( U_{90101} = U_{90102} = 0 \)
Concentrated Loads, \( F \), applied at Nodes 101 & 102

\[ \Rightarrow U_{101} = U_{80101}; \quad U_{102} = U_{80102} \]

Step D: After Lock-off

Element freelnength is introduced back to the mesh
Displacement constraints:
No displacements at nodes 80101 & 80102
Nodes 90101 & 90102 are free to move

\[ \Rightarrow U_{80101} = (U_{101})@step \ C; \quad U_{80102} = (U_{102})@step \ C \]

\[ U_{90101} = U_{101} - (U_{101})@step \ C \]

\[ U_{90102} = U_{102} - (U_{102})@step \ C \]

Figure 2.4.9 - Application of Tieback Lock-off Load for Tieback Aanchors in Clay using ABAQUS
**DEFINITION:**

- $k_1$: Stiffness Representing the Free Length
- $k_2$: Stiffness Representing the Fixed Length
- $k$: Equivalent Overall Stiffness

- $E$: Measured Young's Modulus (from Field Proof Tests)
- $L_t$: Length of Tieback Tendon (free length)
- $A$: Cross-sectional area of the tendon
- $S$: Lateral Spacing of the Tieback
- $L_{ff}$: Length of the Bonded Zone (fixed length)

Assuming

- $k_1 = \frac{EA}{L_t \cdot S}$
- $k_2 = \frac{EA}{[(L_{ff})^3 \cdot S]}$

**Figure 2.4.10** - Calculation of Equivalent Stiffness for Tieback Anchors in Clay
ONE Base Case (with Perfect Struts)

\[ \text{STEP 1} \]

(\textit{Undrained Plane Strain Excavations with } B = 80\text{-}m, \ d_B = 100\text{-}m, \ L = 40\text{-}m \ h = 2.5\text{-}m, \text{ in NC BBC})

SEVEN Cases (with Tieback at 2.5\text{-}m and Perfect Strut at 5.0\text{-}m)

\[ \text{STEP 1} \]

\[ \text{STEP 2} \]

\[ \text{STEP 3} \]

Lock-off Load Considered:

- 49 kN/m
- 98 kN/m
- 147 kN/m
- 196 kN/m
- 245 kN/m
- 392 kN/m
- 490 kN/m

Tieback Properties:

- free length $= 26\text{-}m$
- fixed length $= 12\text{-}m$
- \( E = 1.4E5 \text{ MN/m}^2 \) (20,500 ksi)
- \( A = 10 \text{ cm}^2 \)
- \( S = 4.4\text{-}m \)
- inclined at 22°

(same properties as the tier 1 tiebacks on the South Wall at CA/T ISS 4, see Sec. 4.2)

\[ k_1 = 1.24 \text{ MN/m} \]
\[ k_2 = 5.08 \text{ MN/m} \]

Figure 2.4.11 - Numerical Experiments Illustrating the Effects of Lock-off Load
Figure 2.4.12 - Wall Deflections and Surface Settlements of Excavations with Varying Levels of Lock-off Load at Excavation Depth of 5-m
Figure 2.4.13 - Wall Deflections and Surface Settlements of Excavations with Varying Levels of Lock-off Load at Excavation Depth of 7.5-m
Figure 2.4.14 - Lateral Movements at Tieback Anchorage and Head of the Fixed Length as Functions of Lock-off Force and Excavation Depth
Figure 2.4.15 - Vertical Movements at Tieback Anchorage and Head of the Fixed Length as Functions of Lock-off Force and Excavation Depth
CHAPTER 3

Parametric Analyses of Factors Controlling Excavation-Induced Ground Movements

Finite element analyses for modeling excavations can be placed into two general categories: case histories and parametric studies. Case histories focus on simulating numerically the behavior of actual or proposed excavations at a specific site. Appropriate models and assumptions are used with the goal of properly accounting for the pertinent factors which contribute to the overall excavation performance. Three case studies of excavation in the Boston area are presented in Chapter 4. This chapter focuses on parametric finite element analyses, whose goal is to determine the effects of individual factors influencing the ground reactions to the excavation process.

As described in Chapter 2, the basic analysis tool used in this thesis is the commercially available ABAQUS finite element code with the incorporation of the MIT-E3 soil model [implemented by Hashash, 1992]. The MIT-E3 soil model is capable of describing accurately the behavior of normally to slightly overconsolidated (1.0 ≤ OCR < 8.0) clay specimen under different modes of shearing [Whittle, 1987; Whittle et al., 1994]. In addition to the
implementation and verification of the MIT-E3 soil model in ABAQUS, Hashash [1992] also performed three sets of parametric analyses in order to understand the behavior of braced diaphragm walls in deep deposits of clay. His parametric analyses represent the first attempt at understanding the fundamental behavior of deep excavations in clay [Hashash and Whittle, 1992 & 1996]. The results from these previous studies represent a starting point for this work and are summarized in Section 3.1.

Sections 3.2 to 3.4 present results of new parametric analyses that examine the effects of (1) excavation geometry, (2) soil profile, and (3) support system. Each set of analyses simulate idealized undrained excavations in soil profiles containing Boston Blue Clay (BBC) and modeled using the MIT-E3 soil model input parameters listed in Table 3.1.1. The three sets of numerical experiments can be divided into eight Groups (Groups A to H); Table 3.1.2 summarizes the factors considered in each group of analyses as well as corresponding sections where the results are discussed. Since damage to adjacent structures is directly related to ground deformations, the primary goal of this study is to determine the effect of these factors on ground movements. The forces on the structural elements are also reported in order to provide information on the magnitude of loads on the support system. These observations and findings form the basis for the prediction of ground movements as well as the design recommendations for excavations in clay presented in Chapter 5.

3.1 SUMMARY OF PREVIOUS PARAMETRIC ANALYSES [Hashash and Whittle, 1996]

The previous parametric analyses performed by Hashash and Whittle [1996] evaluated the effects of wall length, support spacing, and stress history
on the behavior of deep excavations in clay. The basic numerical experiment set-up and the results are summarized in Sections 3.1.1. and 3.1.2.

3.1.1 *Model Assumptions*

The primary goal of any parametric study is to isolate and evaluate the effects of specific parameters. In order to capture these effects, some assumptions or simplifications must be made with regards to other factors in the numerical model. The previous parametric study [Hashash and Whittle, 1996] made the following basic assumptions regarding the soil profile, support conditions, and excavation sequence:

1. The subsurface profile consists of 120.0-m deep saturated, low permeability clay. The effective stress-strain-strength properties of the clay are modeled by the MIT-E3 model with input properties corresponding to resedimented Boston Blue Clay (Table 3.1.1). The ground water table is at a depth of 2.5-m and the initial pore pressure is assumed to be hydrostatic [see Figure 3.1.1].

2. The width of the excavation is 40.0-m and the thickness of the concrete diaphragm wall is 0.9-m. This wall is assumed to be "wished-in-place" thus neglecting the possible disturbance effects caused by wall installation. The toe of the wall does NOT extend into the underlying rock but is free to move within the soil mass. The excavation is braced by completely rigid struts (i.e. "perfect struts") which restrict all lateral movements at the locations of the strut after the strut is installed. The excavation is assumed to be infinitely long \( (L_e = \infty) \); therefore, a plane strain condition is assumed.

3. The excavation sequence [See Figure 3.1.2] consists of (1) initial
unsupported excavation of depth $h_u$; followed by (2) additional excavation of $h$ after the installation of a perfect strut located at the top of the excavation tier (i.e., at the height of $h$ above the excavated grade). This step (2) is repeated until failure or desired excavation depth, $H$, is reached. The entire excavation process is assumed to occur rapidly such that there is no migration of pore water; thus, the excavation sequence is modeled as undrained. Since the assumed excavation geometry remain symmetric throughout construction, only half of the excavation is modeled in the FE analyses.

The three factors examined in the previous study were 

1. the wall length [$L = 12.5$-m, 20.0-m, 40.0-m, and 60.0-m$^{31}$],
2. the support spacing [$h = 0.0$-m, 2.5-m, 5.0-m, 7.5-m, and 10.0-m$^{32}$], and
3. the stress history profile [$OCR = 1, 2, 4$, and $C^{33}$].

Table 3.1.3 summarizes the various cases analyzed in this parametric study.

### 3.1.2 Summary of Results

The numerical experiments generated a number of important results concerning the development of excavation-induced ground movements:

1. **Wall Length:** Hashash and Whittle concluded that the wall length does not significantly influence the pre-failure deformations in an excavation. Figure 3.1.3 illustrates the maximum moments, soil movements and wall deflections as function of excavation

---

31 The typical diaphragm wall lengths in the Boston area are in the range of 25-m to 30-m; however, diaphragm walls up to 100-m have been used in practice [Goto and Iguro, 1989].
32 Typical strut support spacing is in the range of 2-m to 4-m.
33 Profile C approximates the stress history profile in the Boston area (see Figure 3.1.7)
depth for excavations in normally consolidated BBC supported by four different lengths \([L = 12.5\text{-m}, 20.0\text{-m}, 40.0\text{-m}, \text{and } 60.0\text{-m}]\) of 0.9-m thick concrete diaphragm wall with support spacing of \(h = 2.5\text{-m}\). The wall deflections and surface settlements corresponding to the 20.0-m and 40.0-m long walls are shown as Figure 3.1.4. The settlement troughs for all wall lengths generally follow similar trends. For excavation depths that are significantly shallower than the failure depth, the observed deformations and moments for all wall lengths virtually coincide; however, as the excavation approaches the failure depth, \(H_s\)\(^{34}\), the deflected wall shapes and magnitudes are highly dependent on the wall embedment depth (i.e. wall length minus the excavation depth). For shorter walls (\(L = 12.5\text{-m} \text{ and } 20.0\text{-m}\)), the maximum lateral deflections develop at the toe of the wall as the excavation approaches the failure depth. In the case of longer walls (\(L = 20.0\text{-m} \text{ and } 40.0\text{-m}\)), the maximum wall deflection remains at 8 to 10-m below the base of the excavation until failure is reached. Therefore, the wall length has limited effects on the pre-failure deformations for excavations in deep layers of clay (which extends below the toe of the wall), but does have a major influence on the location of the failure depth and the failure mechanism of the excavation. The benefit of increased wall length, however, is constrained by the strength of the wall as the moments in the wall increase with excavation depth [Hashash and Whittle, 1996].

2. **Strut Spacing:** Figures 3.1.5 and 3.1.6 illustrate the effects of

\(^{34}\) In the numerical analysis, failure depth is marked as the excavation stage where numerical convergence is not possible due to large incremental soil deformations.
vertical support spacing on deformations and moments in the wall for excavations in normally consolidated BBC supported by 60-m long diaphragm walls with support spacings of \( h = 0.0 \)-m to 10.0-m. The distributions of lateral wall deflections and surface settlements for the "continuous support" case \( h = 0.0 \)-m) and "minimal support" case \( h = 10.0 \)-m) are included in Figure 3.1.5. The maximum lateral wall deflections and maximum bending moments for the five support spacings \( h = 0.0 \)-m, 2.5-m, 5.0-m, 7.5-m, and 10.0-m) considered as a function of excavation depth, \( H \), are shown in Figure 3.1.6. The support spacing has the following three general effects: 1) the support spacing dictates the lateral wall movements above the excavated grade while deep-seated soil movements are defined by the underlying soil properties (see Figures 3.1.5 and 3.1.6); 2) Although the magnitudes of surface settlements also increase with support spacing, support spacing has limited impact on the location of the settlement trough (see Figure 3.1.5); and 3) The initial unsupported cantilever-type wall movements only influence the initial stages of the excavation as the final system behavior is ultimately dominated by movements in the underlying clay (see Figure 3.1.6). This observation suggests that the superposition of two deflection modes as recommended by Clough et al. [1989] for estimating maximum wall deflections is intrinsically conservative.

3. **Stress History**: The influence of stress history is extracted by performing several numerical analyses of excavation in clay profiles with constant OCR [OCR = 1, 2, and 4], where the undrained shear strength increases linearly with depth. In addition to the three
constant OCR profiles, an additional composite profile (referred to as the "C-Profile"), which approximates the stress history profile in Boston, was also considered in this group of analyses (see Figure 3.1.7). Figures 3.1.8 and 3.1.9 illustrate the effects of stress history. As expected, wall deflections and ground movements are much smaller for profiles with overconsolidated clay (OCR = 2.0 and 4.0) and no failure mechanism was observed for excavation depth up to 40.0-m (Figures 3.1.5 and 3.1.8). In the case of the composite profile, where the upper 25-m of overconsolidated clay crust overlies normally consolidated BBC (see Figure 3.1.7), the ground movements associated with an excavation depth greater than 15-m are strongly influenced by the presence of the underlying soft normally consolidated clay (see Figure 3.1.9). The results of this C-profile demonstrated the importance of the underlying clay properties in defining the deep-seated movements associated with excavations in clay

Hashash and Whittle [1996] summarized the results from these numerical experiments in the form of prototype design charts for estimating lateral wall deflections and ground movements. Figure 3.1.10 shows the design charts for estimating maximum lateral wall deflections, \( \delta_{w(\text{max})} \), as functions of the support spacing, excavation depth, and stress history profile. Failure of the diaphragm wall is incorporated in this design chart by including contours of maximum bending moments, \( M_{\text{max}} \). Other ground movements, such as maximum surface settlements, \( \delta_{v(\text{max})} \), and centerline heave, \( \delta_{hv(\text{max})} \), can be estimated using the following equation:
\[ \Delta_{\text{max}} = H \left[ a e^{bh} + (c + d h) H \right] \]  
\[ \text{[Eq. 3.1.1]} \]

where \( \Delta_{\text{max}} \) \( \delta_{w(\text{max})}, \delta_{v(\text{max})}, \text{and } \delta_{h(\text{max})} \) [mm]

\( h \) support spacing [m]

\( H \) excavation depth [m]

\( a, b, c, \text{ and } d \) dimensional constants, see Table 3.1.4.

These prototype design recommendations were derived from the analyses described in this section which consists of few major assumptions with regards to excavation geometry, excavation duration, soil profile, and support conditions. Consequently, there are a number of limitations associated with the design recommendations described in Figure 3.1.10 and Table 3.1.4; these generic design recommendations should be used with caution for cases that deviates significantly from the assumed conditions.
3.2 EXCAVATION GEOMETRY

This section focuses on the effects of cross-sectional geometry on the excavation performance. As summarized in Table 3.1.2, this first set consists of 3 Groups of analyses: Group A) wall length, \( L \); Group B) excavation width, \( B \); and Group C) depth to bedrock, \( d_B \). Section 3.2.1 outlines the scope of the analyses; and the corresponding results for these three groups are presented in Sections 3.2.2 to 3.2.4.

3.2.1 Scope of Experiments for Excavation Geometry

The soil profile and the initial groundwater condition assumed for this set of analyses are similar to Hashash [1992], as summarized in the previous section. The study of excavation geometry considers a normally consolidated clay profile (OCR = 1) with resedimented BBC properties (Table 3.1.1). The groundwater table is located at 2.5-m below the surface. Clay above the ground water table is fully saturated, and the initial pore pressure is hydrostatic. The excavation is supported by a set of 0.9-m thick, "wished-in-place", reinforced concrete diaphragm walls with rigid supports spaced at \( h = 2.5 \)-m. The wall is assumed to be linearly elastic\(^{35}\) with Young's Modulus, \( E = 2.26 \times 10^4 \) MPa, and Poisson's ratio, \( \nu = 0.15 \). The excavation geometry is symmetric throughout the construction period; therefore, only half of the cross-section is modeled.

Figure 3.2.1a shows the excavation geometry assumed in Hashash's [1992] study, while Figure 3.2.1b describes the numerical experiments for this first set of study. Table 3.2.1 contrasts basic assumptions and the factors considered in this portion of the study with the scope of previous

\(^{35}\) Note that, in practice, the allowable moment for a 0.9-m [3-ft] thick, reinforced concrete diaphragm wall is approximately 1.2 MN-m/m [265 kip-ft/ft].
investigation by Hashash and Whittle [1996]. The shaded region in Table 3.2.1 highlights the parameters of interest in these two cases. As indicated in both Figure 3.2.1 and Table 3.2.1, Groups A, B, and C examine the behavior of excavation with wall length between 25-m to 40-m, excavation width of 15-m to 80-m, and depth to bedrock of 30-m to 100-m.

Over 60 plane strain numerical experiments were performed (Table 3.2.2) for the purpose of extracting the effects of wall length, excavation width, and depth to bedrock. Each experiment involves a sequence of steps which includes an initial cantilever phase ($h_u = 2.5$-m) and subsequent installation of rigid struts uniformly spaced at $h = 2.5$-m intervals\textsuperscript{36}. The results of each group of analyses are summarized and presented in Sections 3.2.2 to 3.2.4 respectively.

### 3.2.2 Group A: Effect of Wall Length

Wall length is one of the principal factors already considered in the studies by Hashash and Whittle [1996]. Previous analyses were carried out for L = 12.5-m, 20.0-m, 40.0-m, and 60.0-m long walls. The results show two distinct failure mechanisms (see Figure 3.1.4). For short walls [L = 12.5-m and 20.0-m] there is a basal failure of the soil beneath with the embedded length of the wall deflects in a cantilever mode with the largest deflections occurring at the toe ("kick-out" mode). For long walls [L = 40.0-m and 60.0-m], the failure mechanism is constrained by bending of the embedded wall length ("bulging mode") and the failure may initiate with the formation of a plastic hinge in the wall.

In practice, most diaphragm walls are between L = 20.0-m to 40.0-m in length and therefore cover an intermediate range of behavior which was not

\textsuperscript{36} Same excavation procedure as Hashash and Whittle, 1993.
studied by Hashash and Whittle [1996]. The first group of experiments in this study (Group A) comprises four analyses with wall lengths of \( L = 25.0\)-m, 30.0-m, 35.0-m, and 40.0-m. All four cases assume \( B = 80.0\)-m and \( d_B = 100.0\)-m\(^{37}\). Other reference parameters include a wall thickness of 0.9-m, reinforced concrete diaphragm wall, rigid supports spaced at \( h = 2.5\)-m, and soil profile consisting of normally consolidated (OCR = 1.0) BBC.

Figure 3.2.2 shows schematically the principal parameters extracted and used to evaluate the performance of each analysis. The parameters can be broadly grouped into deformations (lateral wall deflection, surface settlement, surface horizontal displacements, and heave within the excavation) and force on the structural system (including lateral earth pressures, bracing forces/strut loads, and bending moment of the wall). The following paragraphs describe each of these parameters for the Group A experiments:

Figure 3.2.3 shows the deflected wall shapes for the Group A experiments with excavation depth from 2.5-m (unsupported) to 17.5-m\(^{38}\). At \( H = 2.5\)-m, the walls are unsupported and deform in a cantilever mode with the maximum deflection, \( \delta_{w(\text{max})} = 1.3\)-cm, occurring at the top of the wall. As the excavation progresses and rigid struts are installed, the wall bends beneath the lowest level of supports with maximum wall deflection occurring approximately 9-m below the excavated grade (similar observations were made by Hashash and Whittle, 1992). For \( H \leq 7.5\)-m, the deflected wall shapes are virtually identical for all four wall lengths. This observation supports Hashash's conclusion that wall length has minimal effect on pre-failure

\(^{37}\) Group A experiments assume practical upper limits on both the excavation width and depth to bedrock (see Table 3.2.1).

\(^{38}\) The 25.0-m long wall has a final excavation depth of \( H_5 = 17.5\)-m. \( H_5 = 20.0\)-m marks the final depth for the 30.0-m and 35.0-m long walls. The 40.0-m long wall, as reported by Hashash, reported an final excavation depth of \( H_5 = 22.5\)-m. The results for the other three walls beyond an excavation depth of 17.5-m are not shown in Figure 3.2.5.
deformations.

At $H = 10$-m, the toe of the 25-m long wall begins to kick out with maximum incremental lateral deformations at the toe of the wall. The wall movements for the 30-m, 35-m, and 40-m walls, on the other hand, remain in unison. The toe of the 30-m wall kicks out at $H = 15$-m; however, the two longer walls ($L = 35$-m and 40-m) continue to deform in a bulging mode. Similar to criterion used by Hashash and Whittle [1996], failure depth, is marked as the excavation stage where numerical convergence is not possible due to large incremental soil deformations; and $H_S$ denotes the final depth prior to failure. Based on this definition, the final excavation depth for the 25-m long wall occurs at $H_S = 17.5$-m while the final excavation depths prior to failure for the 30-m, 35-m, and 40-m walls are 20-m, 20-m, and 22.5-m respectively. This difference in mode shape of the wall and failure depth of the excavation demonstrates another conclusion reached by Hashash and Whittle [1996] that the wall length has a significant influence on the failure mechanism for braced excavations.

Although Hashash's two conclusions regarding the wall length effects are significant and shown to be valid, there are additional comments that can be added in terms of wall length effects based on the results of Group A analyses:

1. As shown Figure 3.2.5, there are three distinct mode shapes for the braced wall: i) cantilever where maximum deflection is at the top of the wall; ii) bulging where maximum wall deflection is between the excavated grade and the toe of the wall; and iii) kick-out where maximum deflection is at the toe of the wall. The occurrence of each mode depends on the overall structural support system which consists of the bracing system and the support wall. In fact, each
mode shape is a reflection of the effectiveness of the bracing system and adequacy of the wall embedment depth. Omission of the bracing system results in a cantilever type wall deflection. An effective bracing system and support wall should ensure subgrade bulging. The kick-out mode is an indication that the wall embedment depth is inadequate to resist the underlying soil deformations. The support system not only dictates the mode of wall deformation, it also determines which of the two failure types, identified by Hashash and Whittle [1996], the excavation is likely to experience. The kick-out mode is typically the prelude to a basal heave-type failure mechanism, while failure of the wall in a bulging mode represents a deep-seated failure mechanism within the soil mass. This type of deep-seated failure are typically preceded by bending failure of the wall rather than failure in the soil.

2. The wall deflection mode shape after installation of the bracing system is highly dependent on the wall embedment depth for a given soil type and wall stiffness. For excavations in normally consolidated BBC supported by 0.9-m thick concrete diaphragm wall and perfect struts at 2.5-m spacing, kick-out mode is observed when embedment length \((L - H) \leq 15\text{-m}\); if \((L - H) > 15\text{-m}\), the wall should deform in a bulging mode for excavations in normally consolidated BBC.

3. The magnitude of the wall deflections are identical regardless of the wall length as long as the wall remain in the bulging mode shape (i.e. \(L - H > 15\text{-m}\)). If the wall remains in the bulging mode\(^\text{39}\), the maximum wall deflection is at 9-m below the excavation grade. If toe

\(^{39}\) Assuming excavations is normally consolidated BBC supported by concrete diaphragm wall with perfect struts at 2.5-m spacing.
kick-out occurs, the maximum wall deflection (at the toe of the wall) is typically 10% to 50% higher than that observed in a bulging mode at the same excavation depth.

4. For a given wall stiffness, the bracing system controls the lateral movement at the top of the wall above the excavation grade while movements within the embedded portion of the wall are dependent on the length of wall embedment. However, for soils located more than 5-m below the wall (i.e. depth > L + 5m), the support system has negligible effects on soil movements.

The corresponding surface settlements for the Group A experiments are shown in Figure 3.2.4. These settlement curves possess the same characteristics as the settlement troughs observed by Hashash. With the exception of the initial 2.5-m unsupported excavation, the maximum settlements occur at least 10-m from the wall. As the excavation progresses, the settlement magnitudes increase at all points. The location of the maximum settlement, \( x_{(\text{max})} \), migrates away from the wall with excavation depth, while deflections remain in the bulging mode. However, once kick-out occurs, the maximum settlement reaches a limiting location, \( x_{(\text{max})} = 19\)-m (i.e. \( x_{(\text{max})}/y_{(\text{max})} = 19\)-m/25-m = 0.76 for the 25-m wall at \( H \geq 10.0\)-m). At points greater than 40-m from the wall, the effects of wall length on the settlement troughs are essentially negligible at a given excavation depth.

Figure 3.2.5 summarizes the surface horizontal displacements, \( \delta_{h} \), for the four Group A experiments. Although the wall length has a similar influence on the location and magnitude of the maximum deformation \( (\delta_{h(\text{max})} \text{ at } x_{h(\text{max})}) \), the impact of L is less pronounced compared to previous observations for wall deflections and surface settlements. For \( x > 50.0\)-m, the
wall length has no impact on the surface horizontal displacements.

Figure 3.2.6 summarizes the prediction of heave for the Group A experiments. While the wall deflects in a bulging mode, wall length has negligible effect on the predicted magnitudes and distributions of heave. However, when toe kick-out occurs, there are larger heave movements predicted 10-m from the wall extending outward to 15-m from the centerline of these 80-m wide excavations. These observations reinforce the earlier conclusion that changes in wall length impact only a limited zone of soil around the toe of the wall.

All four numerical experiments in Group A assume that the struts are completely rigid; therefore, they reflect the force necessary to keep the wall in place. The resulting fluctuations in strut loads are plotted as functions of the excavation depth in Figures 3.2.7a and 3.2.7b. Note that strut levels 1 (at 0.0-m) and 2 (at 2.5-m) are installed as the excavation proceeds to from H = 2.5-m to H= 5.0-m. Two effects are observed at this stage: a) the wall tends to rotate about the level 2 strut and consequently cause b) increased compressive force in level 2 strut and tensile loads\(^{40}\) in the level 1 strut. In practice, this tensile force will only occur in cross-lot bracing if there is a tensile connection between the bracing and wale beam\(^{41}\).

Level 3 strut is installed at 5.0-m as the excavation progresses to H = 7.5-m. Similarly, each subsequent tier of excavation involves the installation of one level of strut located 2.5-m above the excavation grade. The behavior of struts at all levels except level 1 are similar: 1) the strut attains the maximum load at installation; 2) the strut load drops to its minimum one step following

---

\(^{40}\) Negative forces represent tensile loads.  
\(^{41}\) Generally, tensile connections are not installed in practice, the tensile loads predicted in Strut 1 is considered unrealistic. More realistic scenarios with regards to strut stiffness and locations are considered in Section 3.4.2.
installation; 3) beyond two steps after the installation, the strut load increases slightly and remains relatively constant; and 4) maximum compressive (positive) load increases with the depth of the strut. This pattern of fluctuation in strut load is likely to be controlled by the strut rigidity, wall stiffness, and the location and timing of strut installation (i.e. struts are installed "at grade"). The wall length has negligible effect on the magnitude and behavior of strut loads if the wall deflects in a bulging mode; however, when wall kick-out occurs, increased strut forces are observed.

Figures 3.2.8 and 3.2.9 show the moment distributions at successive excavation depths from the Group A experiments. The sign convention adopted in these figures is the following: negative moment indicates that the outer fiber of the wall on the excavated side is in tension while the retained side of the wall is in compression. As expected from wall deflections shown in Figure 3.2.3, the wall length only affects the moment distribution when toe kick-out occurs (at H = 10.0-m for L=25m). In general, the bending moment diagrams have maximum and minimum values at the level of the lowest strut and in the embedded wall length. Maximum positive moment is at 5.0-m and 2.5-m below the excavation grade for wall deflections in the bulging mode and kick-out mode respectively. When the wall deflects in a bulging mode; the maximum moment above grade remains relatively constant for H ≥ 10-m, while the maximum moment below grade increases with excavation depth. For walls which kick-out, the maximum moment occurs above grade and increases with excavation depth. These analyses assume that the wall remains perfectly elastic with infinite strength. However, typical allowable moment for a 0.9-m thick reinforced concrete diaphragm wall is approximately 1.2-MN-m/m. For excavation depths H ≥ 10.0-m, the maximum moment is no longer within the allowable moment; therefore, the
strength of the wall becomes a limiting factor in assessing the stability of these excavations.

Figures 3.2.10 and 3.2.11 are two sets of horizontal loads on all four walls at excavation depths of 5.0-m and 15.0-m respectively. Each set of figures include a) the horizontal effective horizontal stress ($\sigma'_h$), b) the pore pressure ($u$), and c) total horizontal stress ($\sigma'_h$). The initial effective horizontal stress, pore pressure, and total horizontal stress are also shown in both figures. The behavior, in terms of horizontal loads on the wall, are virtually identical for the four wall lengths considered. In general, the effective horizontal stress on the retained side of the wall above the excavated grade remains relatively constant, while there is a net increase in both the pore pressure and total horizontal stress. Below the excavated grade, the effective horizontal stress decreases by up to 30 kPa on both sides of the wall, with large reductions in the total stress and pore pressure caused by the release of total vertical stress.

The wall length also shows negligible effect on the magnitudes of these horizontal stresses and pore pressures. Significant differences in magnitudes are observed only when the toe kicks out; however, these differences are restricted to the toe of the wall, on the excavated side.

3.2.3 Group B: Effect of Excavation Width

This section focuses on the effects of excavation width on predicted ground movements and structural forces. Underground transportation corridors in the Boston area include projects with widths ranging from 9-m (an underground railway\textsuperscript{42}) to 61-m (an underground highway\textsuperscript{43}). The most

\textsuperscript{42} See the South Cove Project described in Chapter 4.
\textsuperscript{43} See the South Boston Section of the CA/T Project presented in Chapter 4.
widely used design charts generally incorporate the effects of excavation width in the estimation of the factor of safety against basal heave [Bjerrum and Eide, 1956] or as a multiplication factor in estimating the maximum settlement [Mana and Clough, 1981]. The goal of this section is to assess the impact of excavation width in terms of both magnitude and distribution on ground movements and structural loads.

These results of eight Group B experiments (see Figure 3.2.12 and Table 3.2.2) consider undrained excavations in a deep deposit of normally consolidated Boston Blue Clay, \( d_B = 100\text{-m} \), with rigid supports spaced at \( h = 2.5\text{-m} \), and a \( L = 40\text{-m} \) long, 0.9-m thick concrete diaphragm wall. The eight experiments comprise excavation widths of \( B = 15\text{-m}, 20\text{-m}, 30\text{-m}, 40\text{-m}, 50\text{-m}, 60\text{-m}, 70\text{-m}, \) and \( 80\text{-m} \).

Figure 3.2.13 summarizes wall deflections at an excavation depth, \( H = 17.5\text{-m} \) for the Group B experiments. This figure shows that the excavation width has limited effect on the deflected mode shape of the wall while the maximum wall deflection increases by a factor of 2 as the width is increased from \( B = 15\text{-m} \) to \( B = 80\text{-m} \) (\( \delta_w(\text{max}) = 7.2\text{-cm} \) to 14.4-cm). The point of maximum wall deflection is located below the excavated grade at a depth which increases from \( z(\text{max}) = 7.5\text{-m} \) at \( B = 15\text{-m} \) to \( z(\text{max}) = 9\text{-m} \) at \( B = 80\text{-m} \). This small change in \( z(\text{max}) \) is expected to cause a slight change in the embedment length required for toe kick-out (see Sect: \( \text{n} 3.2.2 \)). In general, narrower excavations \( (B < 40\text{-m}) \) show larger differences in the location of the maximum wall deflection, while \( z(\text{max}) \) remain at approximately 9-m below the excavation grade for wider excavations \( (B \geq 40\text{-m}) \).

Figure 3.2.14 summarizes the maximum wall deflections, \( \delta_w(\text{max}) \) as functions of the widths at selected excavation depths, \( H = 2.5\text{-m} \) to 20.0-m. The maximum wall deflections are non-linear functions of \( B \) with a marked
change in gradient at $B = 40$-m and can be approximated by a bilinear curve. For $B \geq 40$-m, the maximum wall deflections increase approximately linearly with the excavation width with no apparent limit for large $B$ (although deformations will ultimately be bounded for a given value of $d_B$).

Figure 3.2.15 shows the surface settlements for the Group B experiments at $H = 17.5$-m. As the excavation width increases from $B = 15$-m to 80-m, the maximum surface settlement increases from $\delta_v(\text{max}) = 3.5$-cm to 8.0-cm, while its location, $x_v(\text{max})$, increases slightly from $x_v(\text{max}) = 18$-m to 21-m. Similar to the wall deflections, the difference in the location of the maximum settlement, $x_v(\text{max})$, is more pronounced for narrower excavations ($B < 40$-m) than for wider excavations ($B \geq 40$-m).

Figure 3.2.16 shows the maximum settlement, $\delta_v(\text{max})$, as a function of the excavation width for depths $H = 2.5$-m to 20-m. At shallower depths, $H \leq 7.5$-m, the maximum settlement increases almost linearly with B; and at large B, the linear trend is also observed with a marked reduction in slope at $B = 30$ to 50-m.

The excavation width, B, has a similar effect on the magnitudes of horizontal surface displacements in the retained soil (see Figures 3.2.17 and 3.2.18). However, the location of $\delta_h(\text{max})$ increases more significantly form $x_h(\text{max}) = 26$-m at $B = 15$-m to $x_h(\text{max}) = 35$-m at $B = 80$-m. The magnitude of maximum surface horizontal displacement also increases with width from $\delta_h(\text{max}) = 2.6$-cm to 6.4-cm for $B = 15$-m to 80-m at $H = 17.5$-m.

Figure 3.2.19 shows the corresponding vertical heave within the excavation at a depth of $H = 17.5$-m for Group B experiments. The excavation width effects both the distribution and the magnitude of the heave. For narrow excavations ($B \leq 30$-m), the maximum heave is predicted at the centerline of the excavation. For intermediate widths ($30$-m < $B \leq 50$-m),

Page 121
there is a plateau of constant heave extending to B/4 from the centerline. For wide excavations (B > 50-m), the maximum heave occurs within 10-m of the wall and decreases toward the centerline. Figure 3.2.20 shows the maximum heave as functions of the excavation width at depths H = 2.5 to 20-m. For H ≤ 7.5-m, the maximum heave is almost independent of the excavation width. However, as the excavation depth increases, $\delta_{hv(max)}$ is a nonlinear function which decreases with B.

Figure 3.2.21 shows the "equivalent" horizontal pressure calculated by dividing the strut load by the strut spacing of 2.5-m at an excavation depth of 17.5-m as defined by Terzaghi and Peck, [1967]. In general, the excavation width causes only a minor change in the apparent/equivalent earth pressure envelope with the largest variations affecting the tensile loads in the top level struts and the compressive load at the lowest level of struts.

Figure 3.2.22 summarizes the predicted moment distributions in the wall for four of the eight Group B experiments at depths H = 7.5-m and 17.5-m. As the excavation width increases from B = 20-m to 80-m, the maximum bending moment increases by 10% (at H = 7.5-m) to 20% (at H = 17.5-m). As expected, there is minimal effect on the bending moment distribution. It should be noted that all of the these Group B experiments are expected to cause yielding of the wall below the excavated grade for H = 17.5-m ($M_{max} > M_{allow} = 1.2 \text{ MNm/m}$).

Figure 3.2.23 shows the pore pressure (u), effective horizontal stress ($\sigma'_h$), and the total horizontal stress ($\sigma_h$) distributions on the wall for excavation widths of 20-m, 40-m, 60-m, and 80-m at H = 17.5-m. The results show minimal influence of the excavation width on the horizontal stresses and pore pressures whose distribution was described previously in Section 3.2.2 and Figures 3.2.10 and 3.2.11. Figure 3.2.23 confirms the general pattern
of behavior observed from the strut loads shown in Figure 3.2.21 as the apparent earth pressure diagram.

3.2.4 Group C: Effect of Depth to Bedrock

The depth to bedrock, $d_B$, is the third component of the excavation geometry. The preceding numerical experiments (Groups A and B) have assumed a deep clay layer with bedrock located at $d_B = 100$-m which represents a practical upper limit on $d_B$. In practice, however, the clay layer is usually less than 100-m deep. This section presents results of Group C experiments with characteristic geometric parameters listed in Figure 3.2.24. Similar to previous two groups of analyses, Group C experiments involve plane strain, undrained excavations in normally consolidated BBC supported by 0.9-m thick reinforced concrete diaphragm wall with perfect struts spaced at $h = 2.5$-m.

The Group C experiments are subdivided into six subgroups (C1 to C6) and consider three excavations widths ($B = 20$, 40, and 80-m), two wall lengths ($L = 25$ and 40-m), and bedrock depths ranging from $d_B = 30$ to 100-m ($d_B = 30$, 37.5, 50, 75, and 100-m). For example, Group C1 consists of three analyses with $L = 40$-m, $B = 80$-m and three values of $d_B = 50$, 75, and 100-m. Group C2 consists of five numerical analyses with $L = 25$-m, $B = 80$-m, and five different depths to bedrock, $d_B = 30$, 37.5, 50, 75, and 100-m. Groups C3 and C4, C5 and C6 form similar sets of analyses, in terms of L and $d_B$, but with widths $B = 40$-m and 20-m, respectively. Overall, there are 24 analyses in the Group C experiments. In order to clarify the presentation and interpretation of results, Table 3.2.3 provides a mapping of the analyses and figures described in this section.

The wall deflections and the corresponding surface settlements for
Groups C1 to C6 are shown sequentially in Figures 3.2.25 to 3.2.30. In general, the bedrock depth only affects wall deflections below the excavated grade, hence, largest effects can be seen at the toe of the wall. For situations where the excavation width is greater than the depth to bedrock \((B > d_B)\), the magnitude of the wall movement increases with \(d_B\) (see Figures 3.2.25 and 3.2.26). However, the net change in \(\delta_w\) at the toe is generally less than 20\% for all cases considered\(^{44}\). When \(B < d_B\), the bedrock depth has minimal effect on the magnitude of wall deflections (see Figure 3.2.29 and 3.2.30). These observations can also been seen in Figures 3.2.32a and 3.2.32b which summarize the normalized maximum settlements and wall deflections \((\delta_{v(max)}/H\) and \(\delta_{w(max)}/H\)).

The bedrock depth has much greater impact on predictions of the surface settlement troughs. In fact, \(d_B\) mainly influences the distribution of far field settlements. For example, Figures 3.2.26a and 3.2.26b show that as \(d_B\) increases, there is a small but progressive reduction in \(\delta_{v(max)}\) with a small increase in \(x_{(max)}\) (also see Figure 3.2.32). The most pronounced change is in the distribution of the far field settlements. As the location of the rigid base becomes shallower, the tail of the settlement trough tapers off much more rapidly. At \(H = 15.0\text{-m}\) (Figure 3.3.26b), the deflected wall shapes are virtually identical for cases with \(d_B = 50\text{-m}, 75\text{-m},\) and \(100\text{-m};\) however, there are large changes in the rate of settlement decay for \(x > 20\text{-m}\). At \(30\text{-m}\) (i.e. \(x = 2H\)) from the wall, the magnitudes of the settlement for cases with \(d_B = 30\text{-m}, 37.5\text{-m}, 50\text{-m}, 75\text{-m},\) and \(100\text{-m}\) are 25\%, 52\%, 71\%, 79\%, and 80\%, respectively, of the maximum. Similar behavior can be seen for Groups C1, C3, and C4 (in Figures 3.2.25, 3.2.27, and 3.2.28). Changes in the distribution of far field

\(^{44}\) One exception can be seen in Figure 3.2.26a, where the largest wall deflections at \(H = 17.5\text{-m}\) occurs for the shallowest \(d_B = 30\text{-m}\). This behavior reflects onset of failure through toe kickout.
settlements are most pronounced when \( d_B/B < 1 \), but are much smaller when \( d_B/B > 1 \) (e.g. Figures 3.2.29 and 3.2.30; Groups C5 and C6).

The effects of \( d_B \) on settlement distributions are illustrated more clearly in Figures 3.2.31a to 3.2.31b which show the normalized settlement troughs, \( \delta_v/\delta_{v(\text{max})} \) versus \( x \) for the Group C experiments at \( H = 7.5\)-m and 15-m, respectively. For each depth \( H \), the results are subdivided according to excavation width (\( B = 80\)-m, 40-m, and 20-m) and wall length (\( L = 40\)-m and 25-m). For example, the top figure in Figure 3.2.31a shows results for \( B=80\)m (Groups C1 and C2), and compares settlement distributions for \( L = 40\)-m and 25-m over a range of \( d_B \) values at \( H = 7.5\)-m. The wall length has no effect on the settlement distributions for \( H = 7.5\)-m (Figure 3.2.31a) as all of the analyses involve bulging deflection mode of the wall (i.e. \( L-H > 15\)-m). In contrast, at \( H = 15\)-m (top figure in Figure 3.2.31b), there is an offset in \( x_{(\text{max})} \) for walls with \( L = 25\)-m (which exhibit toe kick-out mode as \( L-H < 15\)-m) and those with \( L = 40\)-m (which is in bulging mode). The distribution of the settlement for \( x > x_{(\text{max})} \) depends on \( d_B \) and \( B \) with minimal wall length effects. A comparison of figures within Figure 3.2.31a shows that variations in the settlement distribution are less pronounced at \( B = 20\)-m than at \( B = 80\)-m (for the same range of \( d_B \)). The depth to bedrock has a major effect on the settlement distribution when \( d_B < B \) but minimal influence for \( d_B > B \).

Figures 3.2.32a and 3.2.32b show the normalized maximum surface settlements and wall deflections, \( \delta_{w(\text{max})}/H \) and \( \delta_{v(\text{max})}/H \), as a function of depth to bedrock at excavation depths of 7.5-m and 17.5-m, respectively. The location of the bedrock has negligible impact on the magnitude of the maximum wall deflection; however the surface settlement tend to increase as the location of bedrock becomes shallower.

Figures 3.2.33 to 3.2.38 summarize the Group C predictions of surface
horizontal displacements, $\delta_h$. The patterns of behavior in response to changes in $d_B$ and $B$ bear some resemblance to surface settlement. The parameter $d_B$ has a major influence on the magnitude and location of maximum horizontal displacement ($\delta_{h(max)}$ and $x_{h(max)}$) as well as the distribution of $\delta_h$ in the far field. In general, far field displacements, $\delta_{h(max)}$, and $x_{(max)}$ all increase as $d_B$ increases. The effects are most pronounced for wide excavations ($B > d_B$) and where $d_B \ll 1$ (e.g. $d_B = 30$-m in Figure 3.2.34). Bedrock depth has minimal effect on $\delta_h$ when $d_B > B$ (e.g. Figure 3.2.36, where $d_B/B = 2.5$ to 5.0).

Figures 3.2.34 to 3.2.44 show predictions of the vertical heave inside the excavation for the Group C experiments at $H = 5.0$-m, 7.5-m, 12.5-m, and 17.5-m. For each subgroup (C1 to C6) of experiments, the surface heave increases with the depth to bedrock, $d_B$. For narrow excavations ($B = 20$-m, Figures 3.2.43 and 3.2.44), the predicted heave increases across the width of excavation close to a factor of 2 (at $H = 17.5$-m, $L = 25$-m) as $d_B$ increases from 30-m to 100-m. In contrast, for wide excavations ($B = 80$-m), $d_B$ can control the width of the heave zone. For example, with $L = 25$-m (Group C2, Figure 3.2.40), the heave zone extends approximately 25-m from the wall at $d_B = 30$-m, but covers the entire excavation for $d_B > 50$-m. However, there is much more modest effect on $\delta_{hv(max)}$ which increases by only 40% for these same cases.

Figures 3.2.44 to 3.2.47 summarize the strut loads at excavation depths $H = 2.5$ to 17.5-m for Groups C1 to C3. Moment distribution in the wall at $H = 5$, 7.5, 15, and 17.5-m and horizontal stresses on the wall at $H = 15.0$-m for Groups C1 and C3 are presented in Figures 3.2.48 to 3.2.51. In general, the effects of bedrock depth on structural loads are confined within a zone near and below the excavation grade. In terms of strut loads (Figures 3.2.45 to 3.2.47), changes in the location of the bedrock has minimal effect on the loads.
in upper level struts. However, loads in the lower level struts can vary significantly due to changes in bedrock especially when \( B > d_B \) (see Figure 3.4.45). Variations in the bedrock depth also influences the moment distribution in the wall and the horizontal stresses on the wall; however, these effects (Figures 3.3.48 to 3.2.51) are less noticeable compared to displacements and strut loads, and are generally confined to region at and below the excavation grade.

3.2.5 Summary of Excavation Geometry Effects

The first set of numerical experiments examines how changes in the wall length, excavation width, and depth to bedrock influence the soil movements and the loads on the structural elements. All of the numerical experiments assume plane strain, undrained excavation supported by a set of 0.9-m thick concrete diaphragm wall with rigid supports spaced at 2.5-m. In all cases, the magnitudes of the wall deflections, ground movements, and structural forces are monotonically increasing functions of the excavation depth, \( H \). Table 3.2.4 summarizes the results for this first set of numerical experiments.

The first group of analyses, Group A, examines the effects of wall length, \( L \). Results suggest that the wall embedment length, \( (L - H) \) together with the bracing system, determine the wall deformation shape which can be one of the three possible modes: \( i ) \) cantilever \( (z_{(\text{max})} \) at top of wall); \( ii) \) bulging \( (z_{(\text{max})} \) between excavation grade and toe of wall); and \( iii) \) bulging \( (z_{(\text{max})} \) at toe of wall). For the normally consolidated BBC soil profile considered so far, the transition from mode \( ii \) to \( iii \) occurs when \( (L - H) = 15\)-m. The location of maximum surface settlement, \( x_{(\text{max})} \), can be related to these deformation modes of the wall. Once the wall deflection exhibit mode \( iii \), \( x_{(\text{max})} \) remains a
constant value independent of H. However, the effects of wall length on soil movements are limited to depths close to the toe of the wall and 40-m laterally from the excavation. In addition to defining the magnitude and distribution of soil movements, the wall deformation mode also defines the forces on the structural members.

Group B experiments evaluate the effects of excavation width, B. The width plays a major role in controlling the magnitude of ground displacements (settlement, lateral deflections, and heave inside the excavations) but has limited impact on the overall stability or the mode of wall deflections or distributions of ground movements. Maximum surface settlements and lateral deformations typically increase by a factor of 2 as B increase from 15-m to 80-m while heave decreases with increasing B.

The Group C experiments focus on the depth to bedrock, d_B. This parameter has a major influence on the spatial distribution of ground movements, particularly in the far field. As d_B decreases, the settlement trough contracts laterally with minimal change in the location of the maximum settlement, x_{max}. This result has a major impact on prediction of potential damage to adjacent facilities as well as zone of influence for the proposed excavation.
3.3 Soil Profile

The second set of numerical experiments in this study focuses on the impact of soil profile on excavation behavior. Changes in soil type and/or soil stress history inevitably alter the predicted deformations, structural loads, and potential failure conditions of an excavation. All the analyses performed in the previous sections (Section 3.2) assume that the soil is normally consolidated with properties corresponding to resedimented Boston Blue Clay. Although such profiles do occur in recent soft clay deposits, they are relatively unusual in practice. In order to quantify the impact of soil profile, this section presents results for three types of clay-dominated the soil profile:

Group D - Changes in the clay strength/OCR profile within the dominant clay layer;

Group E - Existence of a cohesionless stratum above the clay layer;

Group F - Presence of a overconsolidated clay crust.

Figure 3.3.1 illustrates these three types (Groups D, E, and F) of variations in soil profile, the corresponding results are presented in Sections 3.3.1, 3.3.2, and 3.3.3.

3.3.1 Group D: Variations in Clay Strength/OCR Profile

The influence of soil stress history on excavation-induced ground movements was one of parameters evaluated by Hashash and Whittle [1992]. Four stress history profiles were selected in this previous study (see Section 3.2.1): OCR = 1.0, 2.0, 4.0, and a composite profile (with OCR 1, 2, and 4 clay) which simulate a typical stress history profile found in Boston. The results show that ground movements resulting from excavations in OCR = 2 and 4 clays are significantly smaller than the deformations from the same excavation in normally consolidated clay (cf. Figure 3.1.9). Results for the
Composite profile showed that predicted ground movements were controlled by the presence of the underlying soft (OCR=1) clay. However, it is rather difficult to interpolate the behavior of excavations for intermediate or more complex stress history profiles, especially for cases with $1.0 < \text{OCR} \leq 2.0$. Therefore, the Group D experiments focus on the behavior of excavations in clay with constant stress history between OCR=1.0 and 2.0. Group D comprises 16 numerical analyses (Figure 3.3.2) with combinations of four constant OCR profiles (OCR = 1.15, 1.25, 1.7, and 2.0)\(^{45}\) and four wall lengths (L = 25-m, 30-m, 35-m, and 40-m). All of the Group D experiments assume B = 80-m and $d_B = 100$-m. As in previous sections, the excavation is supported by a 0.9-m thick concrete diaphragm with rigid supports spaced at $h = 2.5$-m. Input parameters for MIT-E3 soil model are listed in Table 3.1.1.

The behavior of Group D excavations with different stress history profiles are evaluated on the basis of wall deflections, surface settlements, moment distributions in the wall, and strut loads. Surface horizontal displacements, heave inside the excavation, and the stresses on the wall are omitted from this discussion as they add little new information regarding the effects of soil profile. This section focuses on pre-failure deformations and load on structures. Stability of the excavation as a function of clay stress history profile will not be discussed. Figures 3.3.3 to 3.3.10 show the ground movements in terms of wall deflections and surface settlements. The forces in the structural elements are reported in Figures 3.3.14 and 3.3.15 as moment distributions in the wall and strut loads.

Figures 3.3.3 and 3.3.4 show the wall deflections and surface settlements for the four cases with stress history profile of OCR = 1.15 and L = 25-m to 40-

\(^{45}\) In all cases, the groundwater conditions are hydrostatic with water table at depth, $d_w = 2.5$-m. Excavation occurs sufficiently rapid that there is no flow of groundwater and the clay is sheared in an undrained mode.
m at all excavation depths up to $H = 20.0$-m. The predicted wall deflections are similar to behavior previously shown for OCR = 1.0 (cf. Figures 3.2.3 and 3.2.4). The wall length affects a) the transition from bulging (bending below the excavated grade) to the toe kick-out mode; and b) the settlement distribution close to the wall (movement of the location of maximum settlement, $x_{(\text{max})}$, in Figure 3.3.4 can also be linked to the modes of the wall deflection). Figures 3.3.5 to 3.3.10 show similar patterns in wall deflections and corresponding surface settlements for OCR's 1.25, 1.70, and 2.0. As the OCR increases, the maximum wall deflection, $\delta_{w(\text{max})}$, and surface settlement, $\delta_{v(\text{max})}$, both decrease, while effects of wall length become negligible at OCR 2.0.

Figure 3.3.11\textsuperscript{46} is a dimensionless plot showing the maximum wall deflections and surface settlements divided by the corresponding excavation depths ($\delta_{w(\text{max})}/H$ and $\delta_{v(\text{max})}/H$) as functions of OCR and $H$. The results show the following:

1. In general, the ratios $\delta_{w(\text{max})}/H$ and $\delta_{v(\text{max})}/H$ are not constant, but generally increase with excavation depth\textsuperscript{47}. However, for OCR $\geq 2.0$, the ratios do remain approximately constant over the range of excavation depths considered in these analyses. This result confirms previous observations that wall deflections and ground movements can be approximated as linear functions of $H$ for the profiles with OCR $\geq 2.0$, while the non-linear development of $\delta_w$ and $\delta_v$ must be considered for OCR < 2.

2. At a given excavation depth, $H$, the maximum wall deflections and surface settlements are monotonic decreasing functions of the

\textsuperscript{46} This figure include excavation geometry of $L = 40$-m, $d_B = 100$-m, $B = 80$-m, and $h = 2.5$-m. This figure also includes an additional analysis for OCR = 4.0.

\textsuperscript{47} This observation does not apply to the initial cantilever phase at $H = 2.5$-m.
OCR. Small deviations from the normally consolidated state are associated with large changes in ratios $\delta_{w(\text{max})}/H$ and $\delta_{v(\text{max})}/H$. For example, at OCR = 1.15 and $H = 17.5\text{-m}$, the maximum wall deflections and surface settlements are only 70% and 60%, respectively or the values plotted at OCR = 1.0.

3. The maximum wall deflections and surface settlements are closely linked for all combinations of OCR and $H$. The ratio $\delta_{v(\text{max})}/\delta_{w(\text{max})} = 0.32$ to 0.55. At $H = 20\text{-m}$, the largest predicted wall deflection ratio $\delta_{w(\text{max})}/H = 0.93\%$ at OCR = 1.0 decreasing to 0.22% at OCR = 4.0, while the corresponding range of $\delta_{v(\text{max})}/H = 0.54\%$ at OCR = 1.0 to 0.1% at OCR = 4.0.

The second effect of stress history is the influence on the transition in deflection modes of the wall from bulging (below grade bending) to toe kick-out. For normally consolidated BBC, the previous experiments (Group A) shown that the embedded wall length must be greater than $(L - H) > 15\text{-m}$ to prevent toe kick-out, while the maximum deflection occurred at 9-m below the excavated grade. As the clay becomes more overconsolidated, the required wall embedment to prevent kick-out decreases. For OCR = 1.15 and 1.25, the transition occurs at $(L - H) = 12.5\text{-m}$; and for OCR = 1.7 and 2.0, at $(L - H) = 10\text{-m}$. These transition depths can be linked to the depth at which the maximum wall deflection, $y_{(\text{max})}$, occurs. Figure 3.3.12 shows the scaled wall deflection, $\delta_{w}/\delta_{w(\text{max})}$, at four selected excavation depths, $H = 2.5\text{-m}$, $5.0\text{-m}$, $15.0\text{-m}$, and $17.5\text{-m}$ for each of the Group D stress history profiles. Initially, the locations of maximum wall deflection are virtually identical (at $H = 2.5\text{-m}$ and $5.0\text{-m}$). However, as the excavation depth increases, the location of $\delta_{w(\text{max})}$ is closer to the excavated grade for the more overconsolidated
profiles. At $H = 17.5$-m, the location of maximum wall deflection, $y_{(\text{max})}$, is at 9-m below the excavated grade at OCR = 1.0, but only at $y_{(\text{max})} = 6.0$-m below the excavated grade at OCR = 2.

Figure 3.3.13 shows the corresponding scaled surface settlement, $\delta_v/\delta_v(\text{max})$ at $H = 5.0$-m and 17.5-m. The distance of the maximum settlement from the wall increases as the clay becomes more overconsolidated. This trend is the reverse of the migration pattern for maximum wall deflections, and suggests that the relationship between wall deflection and surface settlement is highly dependent on the soil profile.

Figure 3.3.13 also shows that the OCR is closely linked to the far field (tail) distribution of surface settlements. The settlement trough expands as the OCR increases. This results also confirms the important link between stress history profile and prediction of settlement distribution.

Figure 3.3.14 shows the moment distribution in the 40-m long wall at $H = 17.5$-m for the six different clay stress history profiles (OCR = 1.0 to 4.0). The moment diagrams have the same distribution with maximum occurring above and below the excavation grade. Above the excavation grade, the maximum moment is always at the location of the last strut. However, the location of maximum moment below the excavation grade is dependent on the soil stress history profile (as expected, similar to $\delta_w$, the location of $M_{(\text{max})}$ is closer to the grade elevation for more overconsolidated profiles).

Also shown in Figure 3.3.14 are the maximum moments above and below the excavation grade for all excavation depths up to $H = 25.0$-m. The magnitude of the maximum moment in the wall decreases as the clay becomes more overconsolidated. For clay with OCR $\leq 1.25$, the concrete diaphragm wall actually reaches the allowable moment $M_{\text{all}} = 1,200$kN-m/m
before the excavation fails (i.e. $H_w < H_s$). 

The equivalent horizontal stresses (for cases with wall length of $L = 40$-m) on the wall at $H = 17.5$-m calculated based on forces in the rigid supports are shown as Figure 3.3.15. The stresses obtained from the five stress history profiles show very similar trend. The most pronounced difference in the stresses is at the vicinity of the last strut. Significantly higher stresses are reported for the normally consolidated case than the $OCR = 2.0$ case.

### 3.3.2 Group E: Effects of Overlying Cohesionless Material

All the analyses performed thus far assume that the surficial deposit consists only of cohesive soil. However, subsurface profiles are usually more complex with multiple soil strata. This section considers excavations in subsurface profiles that are dominated by underlying cohesive deposits but contain significant overlying cohesionless soils. Although cohesionless materials can possess a wide range of stress-strain-strength behaviors, this section focuses on the effect of the **thickness** of this overlying layer on excavation induced deformations. The Group E experiments were devised by considering typical subsurface soil profiles found in the Boston area.

Figure 3.3.16a shows a typical aggregate soil profile to represent conditions in downtown Boston proposed by Johnson [1989]. This profile consists of six major soil layers with three strata above the main deposit of Boston Blue Clay and two strata below. The total thickness of these six strata (i.e. the depth to bedrock) varied from 18-m to 80-m.

The three possible soil layers above the clay deposit are Miscellaneous Fill (typical thickness of 3 to 9-m), Organic Silt (1.5-m to 9-m), and Outwash

---

48 $H_w$ denotes the final excavation depth before the allowable moment is reached; $H_s$, defined earlier, is the final excavation depth prior to failure in the surrounding soil.
Deposits (0 to 7.5-m). The consistency of the main deposit of Boston Blue Clay (12 to 43-m) varies from stiff at the top of the layer to soft at the bottom of the deposit. This clay layer is underlain by either OutwashDeposit (0 to 3-m) or Glacial Till (1.5 to 9-m). Johnson [1989] also lists approximate ranges for engineering properties of these soil layers (Table 3.3.1).

A comparison of this typical subsurface profile with specific sites mentioned in this thesis is presented in Table 3.3.2. Since the focus of this section is on the impact of the thickness of the overlying non-clay soils on the excavation-induced deformations, only the soil layers above the clay layer (Fill, Organic Silt, and Outwash Deposit) are included in Table 3.3.2. At these five excavation sites in Boston, the subsurface profiles consist of Fill, Organic Silt, and BBC49 with a total thickness ranging from 17.0-m to 42.0-m. In all five cases, the overlying material above the clay consists of man-made fill and organic deposits; the outwash deposit above the BBC described by Johnson [1989] is either non-existent or negligible. In general, the subsurface profiles encountered at these five sites are within the typical range described by Johnson [1989] thus demonstrating that Johnson's profile is rather representative of the downtown Boston region where BBC is the predominant deposit. Consequently, this typical profile provides the guideline for designing Group E experiments.

Based on the profile described by Johnson [1989], the BBC stratum is generally encountered at a depth of 4.5-m to 25.5-m. The five excavation sites in the downtown Boston area (Table 3.3.2) show the top of the clay layer at a depth of 2.1-m to 12.2-m. Using these data, the Group E experiments assume two thicknesses overlying the cohesionless soil, \( d_s = 5.0 \)-m and 15.0-m with

---

49 Given the high stiffness of the underlying Till, the experiments assume a rigid base below the clay.
bedrock at $d_B = 50$-m.

A total of four profiles are used in the Group E numerical experiments: Profiles E1, E2, E3, and E4 (Figures 3.3.16b and 3.3.17). Profile E1 comprises 5.0-m of cohesionless soil overlying 45-m of $OCR = 1.0$ BBC. Profile E2 has 35-m of $OCR = 1.0$ BBC overlain by 15-m of cohesionless soil. Profiles E3 and E4 have similar layer thickness of cohesionless soil as E1 and E2, but assume that the clay is slightly overconsolidated with $OCR = 1.15$. All four Group E experiments assume the same excavation geometry ($B = 40$-m$^{50}$, $L = 25$-m, and $h = 2.5$-m) and sequence (undrained excavation with 2.5-m excavation lifts).

Similar to Groups A to D (where $d_s = 0$), the clay deposit is modeled by the MIT-E3 soil model, while the cohesionless soil is described by a much simpler elasto-plastic model using a Drucker-Prager failure criterion with non-associated flow rule (EP-DP model$^{51}$). The elastic shear and bulk moduli are assumed to be proportional to the effective vertical stress ($\sigma'_v0$) thus reflecting the variations in the properties with depth. The properties are estimated based on field and laboratory test data for the overlying cohesionless soils in the Boston region. The cohesionless soil is assumed to have the following properties:

$$\begin{align*}
\text{total unit weight} & = \gamma_t = 18.9 \text{ kN/m}^3 \quad [120 \text{ pcf}] \\
\frac{\sigma'_{ho}}{\sigma'_v0} & = K_0 = 0.5 \\
\text{Poisson's ratio} & = \nu = 0.3 \\
\text{permeability} & = k = 4.3 \text{ m/day} \quad [1.65 \times 10^4 \text{ ft/sec}] \\
\frac{G}{\sigma'_v0} & = 35 \\
\text{friction angle} & = \phi'_{ps} = 30^\circ
\end{align*}$$

The cohesionless soil layer is fully drained with hydrostatic pore pressure maintained in the retained side ($d_w = 2.5$-m in all cases) and no drainage occurs in the clay stratum. As in the preceding section, the Group E

$^{50}$ Note that the Group E analyses consider a narrower excavation than is used in Group D.
$^{51}$ This EP-DP model assumes zero dilatation at failure.
excavation behavior is interpreted from wall deflection, surface settlement, bending moment distributions, and strut loads.

Figures 3.3.18 and 3.3.19 compare the wall deflections and surface settlements from Profiles E1 and E2 with previous results ($d_s = 0$) results for a normally consolidated (all) clay profile$^{52}$. Corresponding results for Profiles E3 and E4 are reported in Figures 3.3.20 and 3.3.21.

The overlying cohesionless soil has a relatively small influence on the magnitude of the wall deflections$^{53}$. The initial unsupported cantilever movements ($H = 2.5$-m), increase slightly with the thickness of the overlying soil, $d_s$. However, as the excavation progresses into the bulging mode, the maximum wall deflections are dictated by deep seated soil movements and there is very little influence of the overlying non-cohesive soil. The impact of the cohesionless soil is visible again when the wall deformation reaches the toe kick-out mode. Profiles E2 and E4 ($d_s = 15$-m) show slightly larger toe movements than the all clay profiles ($d_s=0$); but Profiles E1 and E3 ($d_s = 5$-m) report the same toe movements as the all clay profile. This difference observed for $d_s = 15$-m can be attributed to the soil composition within the required embedment depth$^{54}$. The presence of the overlying cohesionless soil has no impact on the lateral soil movements below the toe of the wall.

The presence of the cohesionless soil has a greater impact on the surface settlement. Figures 3.3.19 and 3.3.21 compare the surface settlements for the Group E experiments ($d_s = 0$, 5-m, and 15-m). The overlying cohesionless soil affects both the shape of the trough as well as its magnitude.

---

$^{52}$ This case in from Group C4 (see Figure 3.2.24).

$^{53}$ In these numerical experiments, the wall is embedded at least 10-m into the clay. If this value is further reduced, then the deep seated movements and the required wall embedment depth will change.

$^{54}$ Note, the required embedment depth to prevent toe kick-out is ($L - H$) = 15-m for OCR 1 and ($L - H$) = 12.5-m for OCR 1.15.
During the initial cantilever phase, the largest settlement occurs when there is no overlying cohesionless soil. However, as the excavation progresses, this trend reverses. In fact, when \( H < (d_s - 5m) \), the analyses predict smaller settlement as \( d_s \) increases. When \( H = (d_s - 5m) \), similar maximum surface settlement is predicted regardless of \( d_s \). When \( H > (d_s - 5m) \), the presence of the cohesionless layer tend to amplify the surface settlement. This surprising result can be explained by considering contour plots of the vertical displacements, \( u_y \), within the soil mass.

Figures 3.3.22 to 3.3.27 show contour plots of settlements\(^{55}\), \( u_y \), for the six analyses presented in the preceding figures at excavation depths, \( H = 5.0-m, 12.5-m, 15.0-m, \) and \( 17.5-m, \) superimposed on the deformed mesh. As the excavation progresses, for cases with \( d_s = 0 \) (Figures 3.3.22 and 3.3.25), the maximum vertical downward movement occurs within the soil mass at a depth 2.5-m to 5-m above the excavation grade. When cohesionless soil is present, the surface settlements tend to reflect the vertical displacements at the top of the clay deposit (at depth \( d_s \)). This result is more clearly seen by comparing the contours for \( d_s = 0 \) and \( d_s = 15-m \) (Figures 3.3.22 and 3.3.24 for \( OCR = 1 \), and Figures 3.3.25 and 3.3.27 for \( OCR = 1.15 \)). When \( H \leq (d_s - 5m) \), the surface settlements are controlled by deformations within the overlying cohesionless layer that are smaller than those for an all-clay profile. When \( H > (d_s - 5m) \), larger surface settlement reflect maximum vertical settlements occurring at the sand-clay interface. The effects of the cohesionless layer are generally confined to distances less than 30-m from the wall (Figures 3.3.19 and 3.3.21).

Figures 3.3.28 and 3.3.29 report the moment distribution in the wall for

---

\(^{55}\) Note that these plots include contours of downward vertical displacement; therefore, heave (upward vertical movement) is not included in these figures.
the Group E soil profiles at excavation depths of 5.0-m, 12.5-m, and 17.5-m. The moment distributions generally follow the same trend with or without the cohesionless soil. For profiles with normally consolidated clay, the concrete diaphragm wall reaches its allowable moment of 1.2 MN-m/m at an excavation depth of \( H_w = 12.5\)-m and 20-m for underlying clay with OCR 1.0 and 1.15, respectively. In both clay stress history profiles, the strength of the wall is a more critical factor than the overall stability of the excavation. The corresponding strut loads in Figures 3.3.30 and 3.3.31 show almost no effects of the overlying soil layer.

3.3.3 Group F: Effects of Clay Crust

The Group F analyses focus on the third type of soil profile which includes presence of a clay crust, where the pre-consolidation pressure increases towards the surface of the clay (cf. Figure 3.3.1). This situation is common in many postglacial clay deposits [cf. Kenny, 1964].

Figure 3.3.16a shows the typical soil profile encountered in the downtown Boston area. In addition to displaying the possible soil strata in this region, the general consistency of the clay is also included in this figure. The top 2.4-m to 8.6-m of the clay layer consists of stiff clay; the next 4.8-m to 17.2-m are classified as medium clay; and the bottom 4.8-m to 17.2-m is the soft clay. This variation in the consistency of the BBC layer reflects the general trend of stress history profile within the deposit which has the typical component of a clay crust (stiff clay) overlaying the main clay body (soft clay). It should be noted that there are large variations in the stress history profile at sites around the Boston area. The most reliable and extensive stress history data have been obtained at two special test sites in South and East Boston [Haley and Aldrich, Inc., 1993] in the early 1990's as a part of the Central
Artery/Third Harbor Project (CA/T). The field and laboratory test program at
the South Boston Special Test Site included over 70 laboratory 1-D
consolidation tests (conventional oedometer and Constant Rate of strain,
CRS, consolidation tests) which established a rather reliable stress history
profile at the site. Figure 3.3.32 summarizes the measured preconsolidation
pressure profile, $\sigma'_P$, obtained at the South Boston Site [Estabrook, 1991]. The
results show that the overconsolidated clay crust extends to a depth of 28-m,
below which, the soft clay is normally to very lightly overconsolidated ($OCR =
\sigma'_P/\sigma'_{v0} = 1.1$ to 1.2).

In addition to the 1-D consolidation tests, the special test program at
South Boston included laboratory $K_0$-consolidated triaxial compression and
direct simple shear (DSS) tests. The resulting undrained shear strength
profile is shown as Figure 3.3.33. The undrained shear strength decreases
with depth in the crust from a maximum $s_{udSS} = 100$ kPa at the top of the
layer to a minimum $s_{udSS} = 50$ kPa at 28-m. Below this depth, the strength of
the soft clay increases linearly with depth to approximately $s_{udSS} = 80$ kPa at
the base of the layer. The undrained shear strengths in triaxial compression
are 30% to 80% higher than those measured in the DSS mode of shearing.$^{56}$

The proposed four profiles in Group F numerical experiments are
based on 1-D consolidation test data obtained from the South Boston Special
Test Program. Figure 3.3.34 shows the four stress history profiles$^{57}$ (Profiles F,
F2, F3, and F4) considered in the Group F analyses as well as the measured

---

$^{56}$ The undrained shear strengths predicted by MIT-E3 using material properties in Table 3.1.1
and the measured stress history profile (Figure 3.3.32) tend to overestimate the measured
strengths in Figure 3.3.33. This is due to the differences in the normalized strength properties
measured on natural BBC and those measured on resedimented BBC (Table 3.1.1). This issue
will be addressed in Chapter 6.

$^{57}$ Each element within the FE mesh has depth $\Delta z = 2.5$-m. A constant OCR value is assumed
within each element. Therefore, the four proposed profiles are step functions with height $\Delta z =
2.5$-m.
OCR data from the South Boston Special Test Site. All four profiles assume that the clay crust has a thickness of 12.5-m. Profiles F1, F2, and F3 assume 15-m of overlying cohesionless soil, while Profile F4 assumes a minimum 5-m thick of overlying cohesionless soil. In all cases, the rigid based is located at \( d_B = 50\)-m.

Profile F1 represents the lower band of the measured OCR profile with normally consolidated (OCR = 1) clay below the crust. Profile F2 corresponds to the mean of the measured data; in this case the lower clay is slightly overconsolidated with OCR = 1.15. Profile F3 assumes a much lower OCR in the crust (maximum OCR \( \approx 2.4 \text{ at } \text{d}_s = 15\)-m), while the underlying clay is normally consolidated (OCR = 1). Profile F4 represents the weakest of the four profiles with normally consolidated BBC below a depth of 17.5-m, and a maximum OCR \( \approx 2.4 \text{ at the top of clay } (\text{d}_s = 5\)-m).

The excavation geometry and sequence are identical to the Group E analyses with B = 40-m, L = 25-m, and rigid struts spaced at \( h = 2.5\)-m. It should be noted that the toe of the wall is located within the clay crust except in the F4 analysis.

The clay deposit is modeled by the MIT-E3 model with the input parameters listed in Table 3.1.1. The cohesionless soil is described by an elasto-plastic material using a Drucker-Prager failure criterion with non-associated flow rule (EP-DP model, cf. Section 3.3.2). Hydrostatic pore pressures are maintained within the cohesionless soil layer on the retained side to simulate drained shearing, while there is no generation of pore water in the underlying clay stratum.

The behaviors of Group F excavations with the four different soil profiles (Profiles F1, F2, F3, and F4) are evaluated on the basis of wall deflections, surface settlements, moment distribution in the wall, and strut
loads. Figures 3.3.36 to 3.3.41 summarize the predicted wall deflections and surface settlements. Figures 3.3.36, 3.3.38, and 3.3.40 compare the wall deflections for Profiles F1 vs. F2, F1 vs. F3, and F3 vs. F4, respectively. In each case, the development of maximum wall deflections \( \delta_{w(\text{max})} \) are compared with results for constant OCR soil profile with overlying cohesionless soil (Group E experiments, Section 3.3.2) as function of excavation depth, \( H \).

All four Group F experiments have the following common features: i) the clay crust has no effect on the magnitude of the initial cantilever deflections (unsupported excavation at \( H = 2.5\)-m); ii) beyond the initial cantilever movement, the presence of the clay crust reduces the magnitude as well as the shape of the wall deflection if the toe of the wall is embedded within the clay crust; and iii) if the toe of the wall is within the underlying soft clay, as the excavation depth increases, the maximum wall deflection tend toward the response predicted for a profile \( OCR = OCR_{\text{min}} \) where \( OCR_{\text{min}} \) is the OCR of the underlying soft clay.

Figures 3.3.36, 3.3.38, and 3.3.40 compare the wall deflection due to three possible variations in the clay crust: 1) slight shift of OCR within the crust and the underlying soft clay (F1 vs. F2, Figure 3.3.36); 2) large variation of OCR within the clay crust with the same underlying soft clay (F1 vs. F3, figure 3.3.38); and 3) shift in the depth of the clay crust within the profile (F3 vs. F4, Figure 3.3.40).

The results from variations 1 and 2 show that the effects of uncertainties in the OCR of the underlying soft clay (F1 vs. F2, Figure 3.3.36) have a much larger effect on the predicted wall deflections than large variations of OCR within the crust itself (F2 vs. F3, Figure 3.3.38). This observation is consistent with the results obtained from the Group D analyses which concluded that small deviations from the normally consolidated state
yield significant decrease in deformation; and the rate of decrease in
displacements also decreases with OCR (see Figure 3.3.11). In terms of
deflected shape, the presence of the clay crust has a slight effect on the depth
of excavation necessary for toe kick-out to occur; however, it is interesting to
note that the largest lateral deformations (when the wall is embedded within
the clay crust) occur at the top of the soft clay, below the toe of the wall.

In fact, the location of the clay crust relative to the toe of the wall is the
most significant of all three factors considered in the Group F experiments
(see Figure 3.3.40, F3 vs. F4). Results for Profile F4, where the crust is located
within the mid-span of the wall, closely resemble the results obtained from
Profile E2 (which does not have a clay crust). Based on results from this
Group F analyses, the location and OCR of the soft clay layer has the most
significant effect on wall deflection; and the actual OCR profile within the clay
crust only has secondary effects on wall deflection.

Figures 3.3.37, 3.3.39, and 3.3.41 show the corresponding surface
settlements for the Group F experiments. The clay crust has a major effect on
the maximum surface settlements but minor impact on the location, \( x_{(\text{max})} \),
or the distribution of settlements beyond 60-m from the support wall. The
predicted maximum surface settlements follow the same trends as maximum
wall deflections. Similar to its effects on \( \delta_w \), variations of OCR within the
crust (F1 vs. F3, Figure 3.3.39) have relatively little effect on predicted \( \delta_{\text{v(max)}} \)
compared to uncertainties in the OCR of underlying soft clay (F1 vs. F2,
Figure 3.3.37). Results for Profile F4 converges to previous solution for
Profile E1 (without clay crust) for \( H > 17.5\)-m.

The distributions and maximum wall bending moments from the
Group F analysis are shown in Figure 3.3.42, at excavation depths \( H = 5\)-m,
12.5-m, and 17.5-m. Similar to previous results, the magnitudes of the
movements are proportional to the wall deflection. Profile F4, with the largest wall deflections, also predicts the largest moment at all excavation depths. Profile F2, on the other hand, generated the smallest moments as well as the smallest wall deflections. Profiles F1, F3, and F4 exceeded the allowable moment of 1.2 MN-m/m in the wall before overall failure in the excavation is encountered ($H_w < H_s$). Similar to previous results, the zone of maximum moment is in the vicinity of the last strut above the excavation grade. Below the excavation grade, the moment in the wall is well below the allowable moment.

The strut loads for the four profiles are shown in Figures 3.3.43. In general, the strut loads follow the same trends presented in the previous sections with the largest load recorded at the last strut. The most significant difference in the strut loads for these four cases is the magnitude of the load in the last strut. The load in the last strut (2.5-m above the excavation grade) decreases as the soil in the vicinity ($\pm 1.5$-m) of the strut becomes more overconsolidated. This observation is consistent with the previous observation for constant stress history profile (Group D).

3.3.4 Summary of Soil Profile Effects

The second set of the parametric analyses focuses on how the patterns and magnitudes of the soil deformations change as different soil profiles are encountered. Three variations in the soil profile have been considered: 1) constant OCR clay profiles (OCR 1 to 3, Group D, Section 3.3.1); 2) presence of an overlying cohesionless stratum (depth $d_s$, Group E, Section 3.3.2); and 3) presence of an overconsolidated clay crust (Group F, Section 3.3.3). Table 3.3.3 summarizes the results from this second set of numerical experiments.

There are four main effects of OCR changes on the ground movement
predictions: 1) small deviations from the normally consolidated state result in significant reductions in the predicted ground deformations. Deformations observed in a OCR 1.15 profile are 30% to 40% less than if the clay stratum were normally consolidated\textsuperscript{58}. 2) As the OCR increases (i.e. relative wall-soil stiffness reduces), the wall embedment depth required to prevent toe kick-out also reduces from \((L - H) = 15\) m at OCR 1 to \((L - H) = 10\) m at OCR 2. 3) Since the location of the maximum surface settlement, \(x_{(\text{max})}\), is linked to the mode of wall deformation, the OCR also affects the location of \(x_{(\text{max})}\). The Group D experiments show that as the clay becomes more overconsolidated, \(x_{(\text{max})}\) increases (from \(x_{(\text{max})} = 22\) m at OCR 1 to \(x_{(\text{max})} = 30\) m at OCR 2 for \(H = 17.5\) m). 4) Changes in clay stress history also affect the distribution of the surface settlement. As OCR increases, the tail of the settlement trough tend to taper off more slowly.

Group E analyses focus on the effect of an overlying cohesionless layer above the main clay deposit. The thickness and the properties of the cohesionless layer are based on the typical values observed in the Boston region. The four analyses in Group E with different cohesionless layer thicknesses and OCR profiles within the main clay layer show that the overlying cohesionless layer has little impact on the lateral movements of the 0.9-m concrete diaphragm wall. However, the presence of the cohesionless layer does alter the magnitude and the distribution of the surface settlement. When the depth of the excavation is well above the cohesionless layer/clay interface, the presence of the cohesionless layer causes the maximum surface settlement to be less than an all-clay profile. If the excavation reaches the clay stratum, observed settlements exceed the deformations for an all-clay profile.

\textsuperscript{58} Since the same 0.9-m thick concrete diaphragm wall is assumed for all the analyses in Group D, this reduction in the displacements also translates to reductions in the observed moments in the wall.
Nevertheless, the effects of the cohesionless layer on the surface settlement are generally confined to a limited zone within 30-m from the wall; beyond this distance, the settlement is dependent on the underlying soil and the location of the rigid base.

The four Group F experiments investigate the effects of an overconsolidated clay crust based on stress history profiles in South Boston. As expected, the presence of the clay crust reduces the predicted wall deflections and surface settlements. The results show that small perturbations in the underlying soft clay have much more influence on wall deflections and surface settlements than relatively large variations in OCR within the crust\textsuperscript{59}. When the excavation depth exceeds the depth of the crust (i.e. toe of the wall embedded within the underlying soft clay), predictions of wall deformations and surface settlements closely approximate the results for a constant OCR profile with the same OCR as the underlying soft clay.

\textsuperscript{59} Note that this finding is consistent with the results from Group D analyses.
3.4 Structural Support System for Excavations

The third set of numerical experiments focuses on the influence of the structural supports on the performance of excavations. In general, excavation support is provided by a wall and bracing system. Soldier pile and lagging, sheet pile section, soil-mix and soldier pile, drilled piers (secant pile), and reinforced concrete diaphragm walls are example wall type that have been used to support excavations. Bracing systems are either compressive (such as cross-lot members or raker beams) or tensile systems (such as tieback anchors grouted either in the retained fill or underlying rock).

The various types of walls exhibit a significant range of bending stiffness and allowable moment. Support walls composed of soldier piles and sheet piles are generally more flexible and capable of sustaining less load than the more rigid drilled piers and reinforced concrete diaphragm walls. These differences in stiffness and strength also exist in the various bracing systems\(^60\). The goal of this section is to determine the effects of wall bending stiffness and axial stiffness of the bracing system on the behavior of the excavations. Section 3.4.1 presents results of the influence of wall bending stiffness (Group G) while Section 3.4.2 considers the axial stiffness of a compressive bracing system (Group H).

---

\(^{60}\) In addition to inherently different material properties, differences in the installation procedures can also affect the performance of the walls (e.g. cast in-situ diaphragm wall vs. driven sheet piles) and the bracing system (e.g. steel struts vs. tieback anchors). This section focuses on the difference in material properties and will not address installation effects.
3.4.1 Group G: Effects of Wall Stiffness

The preceding numerical experiments have all assumed a 0.9-m thick concrete diaphragm wall with elastic bending stiffness $EI = 1440 \text{ MN-m}^2/\text{m}$. Although it is possible to increase this bending stiffness by increasing the wall thickness or using T-panels, most of the walls used in practice have lower bending stiffness. For example, the typical bending stiffness of sheet pile walls are in the range of $EI = 50$ to $80 \text{ MN-m}^2/\text{m}$. This section assesses the effects of wall bending stiffness on the behavior of the excavation.

The wall stiffness effects are evaluated based on 16 numerical analyses referred to as Group G. Figure 3.4.1 illustrates the basic excavation geometry for the Group G analyses. All the analyses within Group G assume an excavation width $B = 40$-m, depth to bedrock $d_B = 50$-m, and wall length $L = 25$-m with rigid supports spaced at $h = 2.5$-m. The two factors varied within Group G are the wall bending stiffness and soil profile.

As indicated in Figure 3.4.1, Group G is further divided into two parts: Group G1 and G2. Group G1 (see Figure 3.4.2) assumes a normally consolidated (OCR = 1) BBC profile and focuses on the response to changes in wall stiffness from $EI = 1440 \text{ MN-m}^2/\text{m}$ (0.9-m thick concrete diaphragm wall) to $4.3 \text{ MN-m}^2/\text{m}$ (sheet pile section PMA-22). Table 3.4.1 summarizes the properties of the support walls considered in this group of numerical experiments. The excavation sequence as well as the displacement and pore pressure boundary conditions are the same as described in the previous sections.

Group G2, which consists of 13 numerical analyses (see Figure 3.4.11), examines how wall stiffness effects are influenced by the soil profile. Group G2 covers the more realistic (Group F) soil profiles presented in Section 3.3.3 with an overlying cohesionless layer and an overconsolidated clay crust. The
G2 experiments also use a more realistic range of bending stiffness with $EI = 1440$-MN-m$^2$/m to 50MN-m$^2$/m (PZ-27 sheet pile section, Table 3.4.1).

The behaviors of excavation in response to changes in wall stiffness are evaluated on the basis of wall deflections, surface settlements, moment distributions in the wall, and strut loads. Table 3.4.2 summarizes the results presented in each figure.

3.4.1.1 Results of Group G1 Experiments

Figures 3.4.3 to 3.4.6 summarize the wall displacements and surface settlements for the six Group G1 experiments at excavation depths $H = 5.0$-m, 7.5-m, 10.0-m, and 12.5-m, respectively. As the bending stiffness of the wall decreases, there is a pronounced change in the shape of the wall, the maximum wall deflection increases and its location migrates towards the excavated grade. At $H = 7.5$-m (Figure 3.4.4), $\delta_{w(\text{max})} = 2.7$-cm for the concrete diaphragm wall with maximum deflection located at approximately 9-m below the excavated grade while the results for the most flexible sheet pile wall (PMA-22 section) show $\delta_{w(\text{max})} = 8.0$-cm occurring 2-m below the grade. Although there are large difference in the deflected shape of the walls, all of the G1 analyses show similar lateral soil movements below 25-m independent of the wall stiffness.

In addition to influencing the deflected wall shape, changes in the wall stiffness also affect the transition from a sub-grade bending mode to toe kick-out. As the wall stiffness decreases, the required embedment depth reduces and hence, the tendency for toe kick-out to occur is much less. For example, at $H = 12.5$-m (Figure 3.4.6), the concrete diaphragm wall is already transiting to a toe kick-out mode, while three larger sheet pile sections (PZ-38, PZ-27, and PWZ-27) deform in the bulging mode (subgrade bending). Excavations
supported by the two most flexible walls, PDA-27 and PMA-22 sheet pile
dsections have a final excavation depth of $H_s = 10$-m before failure mechanism
occurs in the soil. For the three larger sheet pile wall sections (PZ-38, PZ-27,
and PWZ-27), failure mechanism occurs in soil when $H > 12.5$-m\textsuperscript{61}.

The corresponding predictions of surface settlements for Group G1
experiments are also shown in Figures 3.4.3 to 3.4.6. As the wall stiffness
decreases, the maximum settlement also increases and is located closer to the
wall. However, at distances $x \geq 45$-m, there is almost no influence of wall
stiffness on the settlement trough (for all values of $H$). At $H = 7.5$-m (Figure
3.4.4), the maximum settlement $\delta_v(\text{max}) = 1.5$-cm for the diaphragm wall and
occurs at $x(\text{max}) = 15$-m from the wall. In contrast, $\delta_v(\text{max}) = 4.6$-cm for the
PMA-22 section sheet pile wall and this is located at $x(\text{max}) = 10$-m from the
wall.

Figure 3.4.7 summarizes the maximum wall deflections, surface
settlements, and horizontal displacements as functions of the excavation
depth and wall type. These results confirm the effectiveness of using a wall
with high bending stiffness to control ground movements\textsuperscript{62}. The same
results are re-plotted as iso-depth contours (at constant $H$) versus the bending
stiffness, EI (log scale). As a first approximation, the maximum deflections
decrease linearly with log(EI), except when the excavation depth, $H$,
approaches failure (notably at $H \geq 10$m and EI $< 100$ MN-m$^2$/m).

Figure 3.4.9 shows the bending moments for the Group G experiments
at the excavation depth of $H = 10$-m together with normalized maximum and
minimum bending moments, $M_{\text{max}}/M_{\text{all}}$, at other excavation depths ($M_{\text{all}}$

\textsuperscript{61} Based on Group A results, $H_s = 17.5$-m for excavation in NC BBC supported by $L = 25$
diaphragm wall before failure mechanism develops in the soil.
\textsuperscript{62} Similar to Group D results, the ratio of $d_w/H$, $d_v/H$, and $d_h/H$ increase with excavation
depth for excavations in OCR = 1 BBC.
given in Table 3.4.1). Although the bending moment distributions are similar for all six walls within Group G1, the magnitudes of the bending moments are highly dependent on the wall stiffness. This difference in bending stiffness also explains the smaller bending moments reported at smaller sheet pile sections in spite of the larger lateral wall displacements (see Figure 3.2.8). The results also show that the maximum moment exceeds the allowable value for the diaphragm wall at $H_w = 15.0$-m. PWZ-27 and PMA-22 sheet pile wall sections also exceeded the allowable moment at $H_w = 12.5$-m and 10.0-m, respectively, prior to mobilizing the mechanism of failure in the surrounding clay. In contrast, the PZ-38, PZ-27, and PDA-27 sheet pile wall sections have final excavation depths at $H_s = 12.5$-m, 12.5-m, and 10.0-m, respectively by mobilizing the failure mechanism in soil before reaching the full bending strength of the walls ($H_w > H_s$).

The Group G analyses assume that the bracing is perfectly rigid and therefore, the strut loads reported in Figure 3.4.10 reflect the forces necessary to prevent further wall movement after strut installation. In general, the strut loads decrease as the wall becomes more flexible (see Struts 2 and 3 in Figure 3.4.10). However, when the excavation is close of mobilizing the mechanism of failure of the surrounding normally consolidated BBC, the opposite trend occurs. For example, at $H = 12.5$-m, a much larger strut load occurs in level 5 for the PWZ-27 section than those of the PZ-38 or PZ-27 sheet pile sections.
3.4.1.2 Results of the Group G2 Experiments

The Group G2 analyses evaluate the wall stiffness effects for various soil profiles. Figure 3.4.11 summarizes the cases considered in Group G2. The wall stiffness in this group are within the more typical range of 50 to 80 MN-m²/m for sheet pile wall sections and include the realistic soil profiles (Group F experiments) consisting of a clay crust sandwiched between a top cohesionless soil layer and underlying deposit of soft clay (cf. Section 3.3.3).

Figures 3.4.12 to 3.4.15 show the surface settlements and wall deflections for diaphragm and sheet pile wall sections for each of the four soil profiles (F1 to F4). The deflection mode shapes of the low bending stiffness sheet pile sections (PZ 38 and PZ-27) are strongly affected by changes in the soil profile: for profiles F1, F2, and F3 (Figures 3.4.12 to 3.4.14), clay is overlain by 15-m of cohesionless soil. In these cases, the maximum wall deflection occurs very close to the elevation of the excavated grade while to movements of the walls are very similar to those predicted for the diaphragm wall\(^{63}\). Lateral movements in the underlying soil are not affected by the bending stiffness of the wall. The surface settlement distributions reflect the deflection mode shapes of the wall. As the maximum wall deflection occurs at \(z_{(\text{max})} = H\), so the maximum surface settlement occurs at \(x_{(\text{max})} = H\) (i.e. maximum settlement of SPW is closer to the wall than that predicted for diaphragm wall). The wall bending stiffness has minimal effect on the surface settlement for \(x > 45\)-m.

For Profile F4 (Figure 3.4.15), the cohesionless layer is only 5-m deep. In this case, subgrade bending of the wall generates maximum deflections at approximately 2.5-m to 5-m below the excavated grade. Toe movements are similar to the diaphragm wall until the excavation reaches \(H \geq 12.5\)-m. In

\(^{63}\) Larger toe movements do occur for SPW sections when \(H > 15\)-m (i.e. excavating clay crust).
terms of surface settlements, similar settlements are reported for \( x \geq 45 \text{-m} \). For \( x < 45 \text{-m} \), the sheet pile wall generate larger settlements but only a slight shift toward the excavation in the location of \( x_{(\text{max})} \).

Figures 3.4.16 to 3.4.19 summarize the maximum displacements for different wall types as functions of the excavation depth for the four soil profiles considered. For Profiles F1, F2, and F3 (Figures 3.4.16 to 3.4.18), the differences in maximum wall deflections for \( H = 5.0 \text{-m} \) to \( 15.0 \text{-m} \) are due to the deflected shape with \( \delta_{w(\text{max})} \) occurring in the cohesionless soil. As the excavation progress beyond a depth of \( 15 \text{-m} \), the maximum wall displacement actually occurs near the toe of the wall. Since lateral movements at depths are expected to be similar, beyond \( H = 15 \text{-m} \), the maximum wall deflection of the various wall types follow similar trends. Figure 3.4.16 to 3.4.18 also show the details of the wall deflected shape do not affect predictions of maximum surface settlements and surface horizontal displacements. For \( H > 15 \text{-m} \), maximum ground movements (\( \delta_w, \delta_v, \text{and} \delta_h \)) for the SPW sections are typically 20% to 30% higher than those predicted for diaphragm wall.

For Profile F4 (Figure 3.4.19), the maximum displacements (\( \delta_w, \delta_v, \text{and} \delta_h \)) for the sheet pile wall section PZ-38, become progressively larger as \( H \) increases than those values predicted for the diaphragm wall. At \( H = 15 \text{-m} \), the maximum movements are typically 30% to 50% higher for the sheet pile section PZ-38. Table 3.4.4 summarizes the final excavation depths in the analyses, \( H_s \), which corresponds to the onset of failure mechanisms in soil.

Figure 3.4.20 summarizes the maximum displacements at \( H = 10 \text{-m} \) as a function of wall bending stiffness, \( \log (\text{EI}) \). The results are compared with predictions for an all clay profile with constant OCR 1. As in the preceding Group G1 experiments (Figure 3.4.8), the maximum displacements can be represented as linear functions of \( \log (\text{EI}) \) which decreases with increased wall
bending stiffness. The slope of these functions also vary with soil profile, notable for wall deflections (see Figure 3.4.20). Profile F2 has the stiffest overall soil profile and therefore shows the smallest movements. Profiles F1 and F3 have very similar predictions of maximum deformations, suggesting that uncertainties in the modeling of the clay crust have little impact on the predicted performance of sheet pile wall sections. In contrast, the wall bending stiffness has a large input on $\delta_{w(max)}$ for the F4 profile. At high EI (diaphragm wall), the Profile F4 and OCR =1 analyses gave very similar results; however, much smaller movements occur when SPW sections are used with F4 profile. The slopes of the maximum displacement vs. log(EI) reflect the effectiveness of the different wall types in controlling ground movements. As the surrounding soil becomes softer, the use of stiff walls is a good option in controlling the soil movements; however, in relatively stiff soils, changing the wall stiffness will have a very limited impact on the soil movements.

The moment distributions in the various wall types for excavations in the four soil profiles are presented in Figures 3.4.21 and 3.4.24. Each figure contains the projected moment distribution in the wall at an excavation depth of $H = 10$-m together with normalized maximum and minimum moments ($M_{max}/M_{all}$) as functions of excavation depth. The bending moment diagrams are clearly related to the deflection modes of the walls described previously. As the wall stiffness reduces, there are corresponding reductions in the magnitude of moments. Table 3.4.5 shows that the maximum moments for the two sheet pile wall sections (PZ-38 and PZ-27) are within the allowable moment for all of the excavations considered in this group of numerical analyses. The final excavation depth, therefore, is dependent on the strength of the surrounding soil. In the case of the concrete
diaphragm wall, the moment in the wall exceeded the allowable moment for excavations in normally consolidated BBC, Profile F3, and Profile F4. In these three cases, the final excavation depth is constrained by the strength of the wall rather than the soil strength.

The forces in the rigid struts are presented in Figures 3.4.25 to 3.4.28. The initial compressive forces in the struts (except level) decrease as the wall becomes more flexible. After installation, the forces in the struts undergoes a characteristic one cycle oscillation during the next two excavation steps before reaching a stable equilibrium value. The pattern of the fluctuations in the strut loads are very similar for all wall stiffnesses and generally scatter within a range of 100-kN/m.

3.4.2 Group H: Effects of Strut Stiffness

The second component of the structural support system is the bracing system. All of the numerical experiments performed thus far (Groups A to G) assume that the wall supports are completely rigid and movements of the wall at the wall support locations are not possible after the support is installed. This assumption is rarely valid for actual excavations. This section, therefore, focuses on the impact of strut stiffness on the excavation behavior.

The strut stiffness effects are evaluated based on results from three sub-groups of numerical analyses: H1, H2, and H3, illustrated in Figure 3.4.29. The effects of strut stiffness are introduced by considering typical cross-lot bracing elements (76-cm [30"] diameter, 1.9-cm [0.75"] thick steel pipe) with different lateral spacing (S). The Group H analyses do not consider pro-active construction measures such as prestressing of the struts. All the calculations assume B = 40-m, d_B = 50-m, and L = 25-m with a vertical support spacing h = 2.5-m.
As indicated in Figure 3.4.49, Group H is divided into three parts: H1, H2, and H3. Group H1 (see Figure 3.30) assumes that the surficial deposit is an all-clay profile with constant OCR = 1. The excavations are supported by a concrete diaphragm wall (25-m long, 0.9-m thick, and \( h = 2.5 \)-m), while different strut stiffness are simulated by varying the lateral spacing of the assumed strut section. The six analyses in Group H1 cover a wide range of strut axial stiffnesses from \( k_s = 223.1 \text{ MN/m}^2 \) (\( S = 2.0 \)-m) to 37.2 MN/m\(^2\) (for \( S = 12.0 \)-m). Table 3.4.6 summarizes the strut stiffness for the six cases within Group H1. The strut axial stiffness, \( k_s \), is calculated by using the following equation:

\[
k_s = \frac{E_sA_s}{(B/2)^S}
\]

where
- \( k_s \) spring constant per unit length of the wall [kN/m\(^2\)]
- \( E_s \) Young's modulus of steel [200-GN/m\(^2\)]
- \( A_s \) cross sectional area of the steel strut\(^{64}\) [0.0445-m\(^2\)]
- \( B/2 \) half-width of the excavation [\( B/2 = 20 \)-m]
- \( S \) average lateral spacing of the strut [\( S = 2 \)-m to 12-m]

Group H2 and H3 examine the combined effects of strut stiffness, wall stiffness, and soil profile on excavations with \( B = 40 \)-m and \( d_B = 50 \)-m. Group H2 compares the response for the four Group F soil profiles (F1 to F4), as well as the all OCR 1 profile, supported by a diaphragm wall with either rigid or compressible struts spaced at \( S = 6.0 \)-m (total of 10 analyses). Group H3 comprises a similar set of 10 experiments using a PZ-38 sheet pile wall.

3.4.2.1 Results for Group H1 Experiments

Figures 3.4.31 to 3.4.42 show the surface settlements and wall

\(^{64}\) Assuming a 76.2-cm diameter, 1.905-cm thick [30-in diameter 0.75-in] thick steel pipe strut.
deflections for the six Group H1 experiments at depths \( H = 5.0\text{-m}, 7.5\text{-m}, 10.0\text{-m}, 12.5\text{-m}, 15.0\text{-m}, \) and \( 17.5\text{-m} \). At \( H = 5.0\text{-m} \), with one strut located at 2.5-m\(^6\_), the strut stiffness has negligible effect on the wall deflection. For excavation depths \( H \geq 7.5\text{-m} \), decreases in the strut stiffness causes an increase in the magnitude of both wall movements and surface settlements. These results are consistent with previous observations (from Group H) that most of the ground movements are deep seated (below the excavation grade) and are therefore only moderately affected by the bracing stiffness above the grade. However, the characteristic responses of (1) more pronounced bulge in the wall deflection, (2) a steeper and closer settlement trough, (3) limited zone of influence, and (4) reduction in the overall stability of the excavation, associated with decreased wall stiffness (Group H) are also observed for reductions in strut stiffness. For a diaphragm wall, changes in strut stiffness have minimal effect in determining when the toe of the wall kicks out; but do control the expected failure conditions (\( H_s = 17.5\text{-m} \) for perfect struts but decreases to \( H_s = 12.5\text{-m} \) for \( S = 6.0\text{-m} \)).

Figure 3.4.33 shows the maximum displacements plotted as functions of the excavation depth for the six Group H1 experiments as well as the rigid support case. These results show clearly the small effect of bracing stiffness for \( H < 7.5\text{-m} \). Figure 3.4.34 show the same maximum displacement data plotted as a function of the strut stiffness, \( k_s \). As shown in this figure, the strut stiffness effects increase as excavation progresses. For \( H \leq 7.5\text{-m} \), the strut stiffness has negligible impact in controlling the maximum displacements. As the excavation progresses, the role of strut stiffness

\(^{65}\) The initial cantilever wall deflections (\( H = 2.5\text{-m} \)) are not shown in Figure 3.4.31 since they are identical to the movements reported previously for the rigid support case.

\(^{66}\) In previous experiments (Groups A to H), the top strut is located at 0.0-m. In Group G analyses, this top strut (at 0.0-m) is omitted since tensile loads were reported in previous experiment.
becomes more crucial as reflected by the increased slope in Figure 3.4.34 for $H > 12.5$-m.

The distributions of moments in the diaphragm wall at an excavation depth of $H = 12.5$-m are shown in Figure 3.4.35 for the six Group H1 experiments and the rigid bracing analysis. As expected, the stiffness of the bracing system causes changes in the bending moments. Though all seven analyses exhibit a similar pattern of negative moments above grade and positive moments below grade, the magnitudes are strongly influenced by the strut stiffness. Decreases in stiffness are generally accompanied by a reduction in the moment above grade, an increase in moment below grade, and an upward shift in the location of maximum moments. The maximum moments for the rigid supports and $S = 2.0$-m cases occur above grade and exceed the allowable moment for $H_w = 15$-m. For the most compressible bracing case ($S = 12.0$-m), the maximum moment occurs below grade and exceeds the allowable moment at $H_w = 12.5$-m (see Table 3.4.8).

The observed strut loads for Group H1 analyses are reported in Figure 3.4.36 as a function of excavation depth. Strut 1, located at 0.0-m, exists only for the rigid strut case; this surface level strut was omitted for all "real" strut analyses since strut is rarely place at 0.0-m in practice. For the level 2 struts, it should be noted that the tensile (negative) loads reported at $H > 7.5$-m for $S = 2$ and 4-m, and $H > 10$-m for $S = 6$, 8, and 10-m are unrealistic in practice since cross-lot bracing is designed for compressive load not tensile loads. Nevertheless, the following observations are valid for the remaining levels of struts. The initial loads in each strut generally reflect the relative stiffness of the struts. Stiffer struts are accompanied by larger initial strut forces. As the strut becomes more compressible, the fluctuations in the strut load in response to the subsequent excavations also decrease. In the case of the rigid
strut, the initial support force is significantly greater than the "real" struts; and it is always followed by a large drop in the strut force in the next step which is not observed in the "real" strut cases. With the exception of $S = 10$-m and $S = 12$-m cases, the observed strut forces are within the allowable strut loads for the assumed strut section and strut spacing (see Table 3.4.8).

3.4.2.2 Results for Group H2 Experiments

The Group H2 analyses compares the effects of soil profile (Profiles F1 to F4) for a diaphragm wall (Figure 3.4.37) braced by either rigid struts or compressible struts with lateral spacing of $S = 6.0$-m. Figure 3.4.38 to 3.4.43 show the predicted surface settlements and wall deflections for these Group H2 experiments. In all cases, strut rigidity reduces the wall deflections and causes the maximum wall deflection to occur at greater depth below excavated grade. This latter effect also causes the maximum settlement to occur further from the wall (i.e. larger $x_{(max)}$ for more rigid struts). As in the preceding H1 analyses, strut stiffness does not effect the transition depth for wall kick-out but affect significantly the excavation stability. Table 3.4.9 shows the decrease in final excavation depth associated with finite strut compressibility. Approximately 2 to 3-cm of additional wall movement and 1 to 2-cm of surface settlement can be attributed to the use of compressible struts for bracing regardless of soil profile. This observation is illustrated in Figure 3.4.43 which summarizes the maximum displacements as a function of excavation depth for the ten H2 experiments.

Figure 3.4.44 compares the moment distributions in the diaphragm wall for Group H2 analyses at $H = 12.5$-m together with the maximum and minimum moments as function of $H$. These results show that the maximum moments for the compressible strut cases occur below the
excavation grade regardless of the soil profile. Profiles F1, F2, and F3 show relatively small bending moments as the wall is embedded within the crust. For excavations with "real" struts, the moments in the wall are within the allowable moments for a diaphragm wall at all excavation depths (see Table 3.4.9).

The strut loads for Group H2 are shown in Figure 3.4.45\textsuperscript{67}. Similar to observations from Group H1 experiments, rigid struts generate much higher initial loads than compressible struts. In general, higher strut loads (except at Strut 2) are observed when the clay becomes more normally consolidated. Excavations supported by real struts generally yield smaller range of fluctuation, especially during the two excavation stages following strut installation. Most of the observed strut forces are within the allowable load of 743 kN/m; however, this allowable strut force was exceeded at Strut 7 (at 15-m) when the excavation reached a depth of 20-m in Profile F1 and F3 (see Table 3.4.9).

\textsuperscript{67} Similar to Group H1 analyses, Strut 1 at 0.0-m is omitted for experiments with compressible struts; and the tensile loads reported at Strut 2 at 2.5-m are not applicable in practice since bracings are designed for compressive not tensile loads.
3.4.2.3 Results for Group H3 Experiments

Group H3 analyses are very similar in scope to Group H2 experiment but use a more flexible PZ-38 sheet pile section instead of a 0.9-m concrete diaphragm wall (see Figure 3.4.46). The surface settlements and wall deflections for ten Group H3 experiments are shown in Figures 3.4.47 to 3.4.51. Due to the flexibility of the sheet pile wall, the strut stiffness has very limited impact on the predicted wall deflections and surface settlements. The use of compressible wall supports causes larger displacements and a slight upward shift in the location of maximum deflection (see Figure 3.4.47). For the OCR = 1 and F4 profiles (Figures 3.4.47 and 3.4.51) where the overlying cohesionless layer is non-existent or relatively unimportant (d_s = 0 or 5-m), significant increases in \( \delta_w \) and \( \delta_v \) only occur as the wall approaches failure. For Profiles F1, F2, and F3, bracing stiffness causes large increases in the wall deflection within the sand stratum with little impact on underlying movements. In all cases, lateral movements below the toe of the wall are not influenced by the bracing system.

Figure 3.4.52 summarizes the maximum displacements as functions of the excavation depth. Similar to previous observations, maximum displacements are expected to increase by 10% to 25% as more compressible struts are used.

The moment distribution in the wall at an excavation depth of H = 12.5-m are reported in Figure 3.4.53. The maximum and minimum moments at other excavation depths are also reported in this figure. In contrast to calculations for a diaphragm wall (Group H2, Figure 3.4.44), there is very little effect of bracing stiffness on the bending moment diagrams for the sheet pile wall sections. The magnitudes of the moments are very similar for both rigid and compressible bracing and are well below the allowable moment for this
PZ-38 sheet pile section (see Table 3.4.10).

The strut loads for Group H3 are summarized in Figure 3.4.54. Similar to previous observations, when bracing compressibility is included, there are smaller fluctuations in the strut load as the excavation progresses. Smaller strut loads are also observed as the soil profile becomes more overconsolidated. However, unlike results obtained for the more rigid diaphragm wall, the observed forces for the perfect and real struts are rather similar in magnitude and are generally within 50 kN/m from each other at final excavation depth. The strut loads for all cases considered are within the allowable level at all excavation depths (see Table 3.4.10).

3.4.3 Summary of Effects of Wall Stiffness and Strut Stiffness

The third set of experiments target the effect of the stiffness of the support structure. Two Groups of analyses are included in this third set of experiments: Group G and H. Table 3.4.11 summarizes the results from these experiments.

The goal of Groups G1 and G2 numerical experiments is to determine the effects of wall stiffness on excavation behavior. Group G1 considers the impact of wall stiffness for a constant OCR = 1 clay profile; and Group G2 attempts to generalize the results by considering four other soil profiles in addition to the varying wall stiffness. The Group G analyses suggest that the change in wall stiffness generally affect displacements over a limited area. For excavation depths H ≤ 15-m, horizontal displacements below a 25-m long wall and surface movements at x > 45-m, are not affected by wall type. As the wall stiffness decreases, there are pronounced changes in the deflected shapes characterized by larger wall deflections that occur closer to the excavated

---

68 These are the four soil profiles considered in Group F experiments (Section 3.3.3).
grade. The analysis also predict maximum surface settlement that are typically 20% higher for sheet pile wall sections and occur closer to the excavation. The magnitudes of the maximum displacements are related to the wall bending stiffness (Figure 3.4.8). The predicted maximum displacements ($\delta_w$, $\delta_v$, and $\delta_h$) decrease approximately linearly with log(EI). These results demonstrated that the stiffer walls are more effective in reducing movements for excavations in soft soils but have limited benefits in stiffer soils (Figure 3.4.20).

In terms of loads on the structural elements, decreases in wall stiffness generally translate into reduction in the moment in the wall and the initial strut loads. Both the moments in the wall and the fluctuation in the strut loads follow similar trend regardless of the wall stiffness.

The effects of bracing stiffness on excavation performance are illustrated using Groups H1, H2, and H3 analyses. Group H1 compares results for a range of strut stiffness using the constant OCR = 1 clay as the reference soil profile and a 0.9-m concrete diaphragm wall. Group H2 extends the calculations to include a range of soil profiles (F1 to F4) while Group H3 performs similar calculations with a PZ-38 sheet pile section as the support wall. In general, reductions in strut stiffness cause increases in the wall deflection occurring above the excavation grade with the maximum wall movements occurring much closer to the excavated grade. Lateral movements below the toe of the wall are not influenced by the support conditions of the wall. Similar effects are also observed in the surface settlement. As the wall supports become more compressible, the magnitude of the maximum surface settlement increases with negligible movement of the trough location. Settlements at $x > 40$-m are generally unaffected by the bracing stiffness. The reduction in the strut stiffness also reduces the stability
of the excavation by causing the failure mechanism in the soil to develop at a shallower excavation depth.

Changes in the stiffness of the struts also affect the forces on the structural elements. In general, the magnitude of the moments in the wall above the excavation grade tend to decrease as the struts become more compressible. Below the excavation grade, the magnitude of the moments generally increase. Each wall section needs to be evaluated individually to determine if these changes are within the allowable loads. The strut loads are also dependent on the strut stiffness. The initial strut load is higher for the more rigid (increasing $k_s$) struts. The stiffer struts also generate larger fluctuations in the loads in the subsequent excavation stages.
<table>
<thead>
<tr>
<th>Test Type</th>
<th>Parameter / Symbol</th>
<th>Physical Contribution / Meaning</th>
<th>Boston Blue Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. 1-D consolidation oedometer, CRS, etc.</td>
<td>$e_0$</td>
<td>Void ratio at reference stress on virgin consolidation line</td>
<td>0.957</td>
</tr>
<tr>
<td>2. 1-D consolidation oedometer, CRS, etc.</td>
<td>$\lambda$</td>
<td>Compressibility of virgin OCR=1 clay</td>
<td>0.184</td>
</tr>
<tr>
<td>3. 1-D consolidation oedometer, CRS, etc.</td>
<td>$C$</td>
<td>Nonlinear volumetric swelling behavior</td>
<td>22.0</td>
</tr>
<tr>
<td>4. 1-D consolidation oedometer, CRS, etc.</td>
<td>$n$</td>
<td>Nonlinear volumetric swelling behavior</td>
<td>1.6</td>
</tr>
<tr>
<td>5. 1-D consolidation oedometer, CRS, etc.</td>
<td>$h$</td>
<td>Irrecoverable plastic strain</td>
<td>0.2</td>
</tr>
<tr>
<td>6. $K_0$ Oedometer or $K_0$ triaxial</td>
<td>$K_{0nc}$</td>
<td>$K_0$ for virgin normally consolidated clay</td>
<td>0.53</td>
</tr>
<tr>
<td>7. $K_0$ triaxial</td>
<td>$2G/K$</td>
<td>Ratio of elastic shear to bulk modulus (Poisson’s ratio for init. unloading)</td>
<td>1.05</td>
</tr>
<tr>
<td>8. Undrained triaxial shear tests, OCR=1; $CK_0UC$</td>
<td>$\phi'_{TC}$</td>
<td>Critical state friction angles in triaxial compression</td>
<td>33.4°</td>
</tr>
<tr>
<td>9. Undrained triaxial shear tests, OCR=1; $CK_0UE$</td>
<td>$\phi'_{TE}$</td>
<td>Critical state friction angles in extension (large strain failure criterion)</td>
<td>45.9°</td>
</tr>
<tr>
<td>10. Undrained triaxial shear tests, OCR=1; $CK_0UC$</td>
<td>$c$</td>
<td>Undrained shear strength (geometry of bounding surface)</td>
<td>0.866</td>
</tr>
<tr>
<td>11. Undrained triaxial shear tests, OCR=1; $CK_0UC$</td>
<td>$S_1$</td>
<td>Amount of post peak strain softening in undrained triaxial compression</td>
<td>4.5</td>
</tr>
<tr>
<td>12. Undrained triaxial shear tests, OCR=2; $CK_0UC$</td>
<td>$\omega$</td>
<td>Nonlinearity at small strains in undrained shear</td>
<td>0.07</td>
</tr>
<tr>
<td>13. Undrained triaxial shear tests, OCR=2; $CK_0UC$</td>
<td>$\gamma$</td>
<td>Shear induced pore pressure for OC clay</td>
<td>0.5</td>
</tr>
<tr>
<td>14. Resonant Column or shear wave velocity from cross-hole tests</td>
<td>$\kappa_0$</td>
<td>Small strain compressibility at load reversal</td>
<td>0.001</td>
</tr>
<tr>
<td>15. Drained triaxial</td>
<td>$\psi_0$</td>
<td>Rate of evolution of anisotropy (rotation of bounding surface)</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 3.1.1 - Input Parameters for Boston Blue Clay Using MIT-E3 Model (after Whittle, 1987, 1990)
<table>
<thead>
<tr>
<th>Factors Considered</th>
<th>Section</th>
<th>Analysis Group</th>
<th>Variable</th>
<th># of analyses</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>I. Excavation Geometry</strong></td>
<td>Section 3.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wall Length, L</td>
<td>3.2.2</td>
<td>Group A</td>
<td>L</td>
<td>[4]</td>
</tr>
<tr>
<td>Excavation Width, B</td>
<td>3.2.3</td>
<td>Group B</td>
<td>B</td>
<td>[8]</td>
</tr>
<tr>
<td>Depth to Bedrock, dB</td>
<td>3.2.3</td>
<td>Group C</td>
<td>dB</td>
<td>[24]</td>
</tr>
<tr>
<td><strong>II. Soil Profile</strong></td>
<td>Section 3.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>OCR Profile</td>
<td>3.3.1</td>
<td>Group D</td>
<td>OCR</td>
<td>[16]</td>
</tr>
<tr>
<td>Cohesionless Layer</td>
<td>3.3.2</td>
<td>Group E</td>
<td>d_s</td>
<td>[4]</td>
</tr>
<tr>
<td>Clay Crust</td>
<td>3.3.3</td>
<td>Group F</td>
<td>Profiles F1 to F4</td>
<td>[4]</td>
</tr>
<tr>
<td><strong>III. Support System</strong></td>
<td>Section 3.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wall Stiffness</td>
<td>3.4.1</td>
<td>Group G</td>
<td>Ei</td>
<td>[16]</td>
</tr>
<tr>
<td>Strut Stiffness</td>
<td>3.4.2</td>
<td>Group H</td>
<td>k_s</td>
<td>[25]</td>
</tr>
</tbody>
</table>

Table 3.1.2 - Summary of Factors Analyzed in the Parametric Study

<table>
<thead>
<tr>
<th>Parameters Examined by Hashash and Whittle [1993]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Wall Length [L]</strong></td>
</tr>
<tr>
<td>L = 12.5-m [41.0-ft] (short)</td>
</tr>
<tr>
<td>L = 20.0-m [65.6-ft] (medium)</td>
</tr>
<tr>
<td>L = 40.0-m [131.2-ft]</td>
</tr>
<tr>
<td>L = 60.0-m [196.9-ft]</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

* The composite profile (OCR = C) consists of the following: top 15-m [49.2-ft] with OCR = 4; next 10-m [32.8-ft] with OCR = 2; the remaining layer is normally consolidated.

Table 3.1.3 - Summary of Parametric Study Performed by Hashash and Whittle [1996]
\[ \Delta_{\text{max}} = H[ae^{\Delta_h} + (c + dh)H] \]

<table>
<thead>
<tr>
<th>Stress history profile OCR (1)</th>
<th>Maximum movement (mm)</th>
<th>Eq. (1) Coefficients*</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( a ) (3)</td>
<td>( b ) (m(^{-1})) (4)</td>
<td>( c ) (m(^{-1})) (5)</td>
<td>( d ) (m(^{-1})) (6)</td>
<td></td>
</tr>
<tr>
<td>1.0</td>
<td>Wall, ( \delta_{\text{max}} )</td>
<td>1.7610</td>
<td>0.0184</td>
<td>0.1650</td>
<td>0.0323</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Settlement, ( w_{\text{max}} )</td>
<td>0.2854</td>
<td>0.2216</td>
<td>0.1061</td>
<td>0.0175</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Heave, ( w_{\text{max}} )</td>
<td>3.0449</td>
<td>-0.0244</td>
<td>0.3070</td>
<td>0.0304</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>Wall, ( \delta_{\text{max}} )</td>
<td>2.1972</td>
<td>0.0515</td>
<td>0.0</td>
<td>0.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Settlement, ( w_{\text{max}} )</td>
<td>0.8768</td>
<td>0.0618</td>
<td>0.0</td>
<td>0.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Heave, ( w_{\text{max}} )</td>
<td>4.4012</td>
<td>0.0180</td>
<td>0.0</td>
<td>0.0</td>
<td></td>
</tr>
<tr>
<td>4.0</td>
<td>Wall, ( \delta_{\text{max}} )</td>
<td>1.3878</td>
<td>0.0548</td>
<td>0.0</td>
<td>0.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Settlement, ( w_{\text{max}} )</td>
<td>0.5541</td>
<td>0.0638</td>
<td>0.0</td>
<td>0.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Heave, ( w_{\text{max}} )</td>
<td>2.6961</td>
<td>0.0161</td>
<td>0.0</td>
<td>0.0</td>
<td></td>
</tr>
<tr>
<td>Composite</td>
<td>Wall, ( \delta_{\text{max}} )</td>
<td>0.8557</td>
<td>-0.0984</td>
<td>0.1646</td>
<td>0.0152</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Settlement, ( w_{\text{max}} )</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0941</td>
<td>0.0070</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Heave, ( w_{\text{max}} )</td>
<td>0.0</td>
<td>0.0</td>
<td>0.3841</td>
<td>0.0200</td>
<td></td>
</tr>
</tbody>
</table>

*Coefficients give \( \Delta_{\text{max}} \) (mm) for \( H \) (m), \( h \) (m).

Table 3.1.4 - Coefficients for Interpolating Maximum Wall and Ground Movements
Hashash and Whittle [1996]
## Factors Influencing Excavation-induced Deformations

### I. Excavation Geometry

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Model Parameters (Hashash &amp; Whittle, 1996)</th>
<th>Model Parameters (for analyzing L, B, and dB: Section 3.2.2, 3.2.3, &amp; 3.2.4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of Excavation, B</td>
<td>40.0-m [131.2-ft]</td>
<td>15.0-m, 20.0, 30.0, 40.0, 50.0, 60.0, 70.0, &amp; 80.0-m [49.2-ft to 262.5-ft]</td>
</tr>
<tr>
<td>Depth to Bedrock, dB</td>
<td>120.0-m [393.7-ft]</td>
<td>30.0-m#, 37.5#, 50.0, 75.0, and 100.0-m [98.4-ft# to 328.1-ft]</td>
</tr>
</tbody>
</table>

### II. Soil Profile

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Model Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay OCR profile</td>
<td>OCR = 1, 2, 4, and C* 1.0</td>
</tr>
</tbody>
</table>

### III. Support System

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Model Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Length, L</td>
<td>12.5-m, 20.0-m, 40.0-m, and 60.0-m [41.0-ft to 196.9-ft]</td>
</tr>
<tr>
<td>Wall Type</td>
<td>0.9-m [3.0-ft] thick concrete diaphragm wall</td>
</tr>
<tr>
<td>Rigid Strut Spacing, h</td>
<td>0.0, 2.5, 5.0, 7.5, &amp; 10.0-m [0.0 to 32.8-ft]</td>
</tr>
</tbody>
</table>

### D. Drainage Condition

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Model Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>undrained</td>
</tr>
</tbody>
</table>

Note: 
- This dB is used for the case of L = 25.0-m only.
- OCR = C is a combined/composite stress history profile obtained by matching the clay strength profile typically found in the Boston region.
- Shaded entries indicate that the factor is included in this parametric study.

Table 3.2.1 - Summary of Principal Parameters in Hashash's Parametric Study and the new Numerical Experiments on Wall Length, Excavation Width, and Depth to Bedrock covered in Sections 3.2.2, 3.2.3, and 3.2.4.
<table>
<thead>
<tr>
<th>Excavation Width, B</th>
<th>Wall Length, L</th>
<th>$d_B=30.0$-m</th>
<th>$d_B=37.5$-m</th>
<th>$d_B=50.0$-m</th>
<th>$d_B=75.0$-m</th>
<th>$d_B=100$-m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[98.4-ft]</td>
<td>[123.0-ft]</td>
<td>[164.0-ft]</td>
<td>[246.1-ft]</td>
<td>[328.1-ft]</td>
<td></td>
</tr>
<tr>
<td>B = 15.0-m</td>
<td>L = 25.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>L = 30.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>L = 35.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>L = 40.0-m</td>
<td>NA</td>
<td>NA</td>
<td>C5</td>
<td>C5</td>
<td>B, C5</td>
</tr>
<tr>
<td>B = 20.0-m</td>
<td>L = 25.0-m</td>
<td>C6</td>
<td>C6</td>
<td>C6</td>
<td>C6</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>L = 30.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>L = 35.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>L = 40.0-m</td>
<td>NA</td>
<td>NA</td>
<td>C5</td>
<td>C5</td>
<td>B, C5</td>
</tr>
<tr>
<td>B = 30.0-m</td>
<td>L = 25.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>L = 30.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>L = 35.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>L = 40.0-m</td>
<td>NA</td>
<td>NA</td>
<td>C3</td>
<td>C3</td>
<td>B, C3</td>
</tr>
<tr>
<td>B = 40.0-m</td>
<td>L = 25.0-m</td>
<td>C4</td>
<td>C4</td>
<td>C4</td>
<td>C4</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>L = 30.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>L = 35.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>L = 40.0-m</td>
<td>NA</td>
<td>NA</td>
<td>C3</td>
<td>C3</td>
<td>B, C3</td>
</tr>
<tr>
<td>B = 50.0-m</td>
<td>L = 25.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>L = 30.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>L = 35.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>L = 40.0-m</td>
<td>NA</td>
<td>NA</td>
<td>B</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>B = 60.0-m</td>
<td>L = 25.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>L = 30.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>L = 35.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>L = 40.0-m</td>
<td>NA</td>
<td>NA</td>
<td>B</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>B = 70.0-m</td>
<td>L = 25.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>L = 30.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>L = 35.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>L = 40.0-m</td>
<td>NA</td>
<td>NA</td>
<td>B</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>B = 80.0-m</td>
<td>L = 25.0-m</td>
<td>C2</td>
<td>C2</td>
<td>C2</td>
<td>C2</td>
<td>A, C2</td>
</tr>
<tr>
<td></td>
<td>L = 30.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
</tr>
<tr>
<td></td>
<td>L = 35.0-m</td>
<td>NA</td>
<td>NO</td>
<td>NO</td>
<td>NO</td>
<td>A</td>
</tr>
<tr>
<td></td>
<td>L = 40.0-m</td>
<td>NA</td>
<td>NA</td>
<td>C1</td>
<td>C1</td>
<td>A, B, C1</td>
</tr>
</tbody>
</table>

NOTE: NA indicates that an analysis is not appropriate because $L \leq d_B$. NO indicates that the analysis was not performed because it was deemed unnecessary. The entries in the shaded regions identify the analysis group and indicates that the results are presented in Section 3.2; blank entries indicates though analyses were performed, results are not presented in Section 3.2.

Table 3.2.2 - Summary of Cases Analyzed in Current Parametric Analyses of of Excavation Geometry [OCR=1.0, $h=2.5$-m, 0.9-m concrete diaphragm wall]
<table>
<thead>
<tr>
<th>Analysis Group</th>
<th>Excavation-induced Deformations</th>
<th>Loads on Structural Elements</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\delta_w$ &amp; $\delta_v$</td>
<td>$\delta_h$</td>
</tr>
<tr>
<td><strong>Group C1</strong></td>
<td>Figure</td>
<td>Figure</td>
</tr>
<tr>
<td>$B=80.0$-m</td>
<td>3.2.25</td>
<td>3.2.33</td>
</tr>
<tr>
<td>$L=40.0$-m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$d_B=50, 75, &amp; 100$-m</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Group C2</strong></td>
<td>Figure</td>
<td>Figure</td>
</tr>
<tr>
<td>$B=80.0$-m</td>
<td>3.2.26</td>
<td>3.2.34</td>
</tr>
<tr>
<td>$L=25.0$-m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$d_B=30, 37.5, 50, 75, &amp; 100$-m</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Group C3</strong></td>
<td>Figure</td>
<td>Figure</td>
</tr>
<tr>
<td>$B=40.0$-m</td>
<td>3.2.27</td>
<td>3.2.35</td>
</tr>
<tr>
<td>$L=40.0$-m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$d_B=50, 75, &amp; 100$-m</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Group C4</strong></td>
<td>Figure</td>
<td>Figure</td>
</tr>
<tr>
<td>$B=40.0$-m</td>
<td>3.2.28</td>
<td>3.2.36</td>
</tr>
<tr>
<td>$L=25.0$-m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$d_B=30, 37.5, 50, 75, &amp; 100$-m</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Group C5</strong></td>
<td>Figure</td>
<td>Figure</td>
</tr>
<tr>
<td>$B=20.0$-m</td>
<td>3.2.29</td>
<td>3.2.37</td>
</tr>
<tr>
<td>$L=40.0$-m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$d_B=50, 75, &amp; 100$-m</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Group C6</strong></td>
<td>Figure</td>
<td>Figure</td>
</tr>
<tr>
<td>$B=20.0$-m</td>
<td>3.2.30</td>
<td>3.2.38</td>
</tr>
<tr>
<td>$L=25.0$-m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$d_B=30, 37.5, 50, 75, &amp; 100$-m</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTE:**
- $\delta_w$: Wall Deflections
- $\delta_v$: Surface Settlements
- $\delta_h$: Surface Horizontal Displacements
- $\delta_{hv}$: Heave within the Excavation
- $M$: Moment Distribution in Wall
- $\sigma'_{h}$: Effective Horizontal Stress
- $u$: Pore Pressure
- $\sigma_h$: Total Horizontal Stress

Each entry identifies the Figure containing the corresponding results. A blank entry indicates that the results are not reported in Section 3.2.4.2.

Table 3.2.3 - Summary of Results of Group C Experiments Presented in Section 3.2.4 -- Evaluation of Depth to Bedrock Effects
<table>
<thead>
<tr>
<th>Wall Length, L</th>
<th>Wall Displacements</th>
<th>Surface Settlements</th>
<th>Horizontal Displacements</th>
<th>Heave</th>
<th>Moments in the Wall</th>
<th>Strut Loads</th>
<th>Horiz. Stresses</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>determines the deflection phase:</td>
<td>no effects on surface settlement if wall remains in bulging mode.</td>
<td>similar effects as surface settlements but less pronounced</td>
<td>negligible impact when wall remains in bulging mode.</td>
<td>negligible impact when wall remains in bulging mode.</td>
<td>negligible impact when wall remains in bulging mode.</td>
<td>negligible impact when wall remains in bulging mode.</td>
</tr>
<tr>
<td>[Group A]</td>
<td>if (L-H) &gt; 15m then bulging mode</td>
<td>kick-out: increase in maximum settlement and shift in location of maximum settlement toward excavation.</td>
<td></td>
<td>kick-out: increased heave movements near the wall.</td>
<td>kick-out: shift in the location of maximum moment and increased moment at the location of lowest level strut are observed</td>
<td>kick-out: increased strut load in the lower levels</td>
<td>kick-out: differences are confined near the excavation grade.</td>
</tr>
<tr>
<td></td>
<td>if (L-H) ≤ 15m, then kick-out mode</td>
<td>no effects on magnitude if wall remains in &quot;bulging&quot; mode.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Excavation Width, B</td>
<td>influences the deflected shape for narrow excavations. magnitude increases with B.</td>
<td>influences the location of the maximum settlement for narrow excavations. magnitude increases with B.</td>
<td>influences the location of maximum horizontal displacement for narrow excavations. magnitude incr. with B.</td>
<td>influences the distribution of moment for narrow excavations. magnitude increases with B.</td>
<td>influences the distribution of moment for narrow excavations.</td>
<td>minor impact on strut loads</td>
<td>minimal influence on horizontal stresses</td>
</tr>
<tr>
<td>[Group B]</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth to Bedrock, dB</td>
<td>limited impact on wall deflection</td>
<td>significant impact on the tail of the settlement for shallow depth to bedrock.</td>
<td>significant impact on distribution and magnitude.</td>
<td>surface heave increases with depth to bedrock.</td>
<td>impact confined to area near the excavation grade.</td>
<td>impact struts located near the excavation grade.</td>
<td>impact confined to area near the excavation grade.</td>
</tr>
<tr>
<td>[Group C]</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.2.4 - Summary of results for the Excavation Geometry Finite Element Experiments
<table>
<thead>
<tr>
<th>Geologic Unit</th>
<th>Description</th>
<th>Saturated Unit Weight kg/m³ (pcf)</th>
<th>Natural Water Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Undrained Shear Strength kg/m² (psf)</th>
<th>Other</th>
<th>Allowable Bearing Pressure kg/m² (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Misc. Fill</td>
<td>Loose to very dense sand, gravelly sand or sandy gravel, intermixed with varying amounts of silt, cobbles or boulders, &amp; miscellaneous brick, rubble, trash or other foreign materials</td>
<td>1600 - 2000 (100 - 125)</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>Organics</td>
<td>Very soft to medium stiff grey clayey organic silt or brown fibrous peat with trace amounts of shells, fine sand &amp; wood</td>
<td>14400 - 1760 (90 - 110)</td>
<td>40 - 100</td>
<td>-----</td>
<td>-----</td>
<td>1465 - 3900 (300 - 800)</td>
<td>Organic Content: 5 - 25%</td>
<td>-----</td>
</tr>
<tr>
<td>Outwash Deposits</td>
<td>Medium dense to dense, brown coarse to fine or medium to fine sand with varying amounts of gravel &amp; silt</td>
<td>1760 - 2160 (110 - 135)</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>19500 - 48800 (4000 - 10000)</td>
</tr>
<tr>
<td></td>
<td>(Boston Blue Clay)</td>
<td>1824 - 1920 (114 - 120)</td>
<td>30 - 40</td>
<td>40 - 55</td>
<td>15 - 30</td>
<td>2930 - 5860 (600 - 1200)</td>
<td>Re-compression Ratio = 0.02 - 0.04</td>
<td>9760 - 19500 (2000 - 4000)</td>
</tr>
<tr>
<td></td>
<td>Soft to very soft, gray silty clay, occasional layers of fine sand or silt</td>
<td>1810 - 1890 (113 - 118)</td>
<td>30 - 50</td>
<td>40 - 55</td>
<td>15 - 30</td>
<td>1950 - 3900 (400 - 800)</td>
<td></td>
<td>4880 - 9760 (1000 - 2000)</td>
</tr>
<tr>
<td>Outwash Deposits</td>
<td>Medium to dense, stratified sands &amp; gravels in discontinuous layers</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td></td>
<td>Variable</td>
</tr>
<tr>
<td>Glacial Till</td>
<td>Dense to very dense heterogenous mixture of sand, gravel, clay &amp; silt with cobbles &amp; rock fragments</td>
<td>2000 - 2240 (125 - 140)</td>
<td>10 - 20</td>
<td>15 - 30</td>
<td>10 - 20</td>
<td>9760 - 39000 (2000 - 8000)</td>
<td></td>
<td>39000 - 98000 (8000 - 20000)</td>
</tr>
<tr>
<td>Bedrock</td>
<td>Cambridge Argillite</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td></td>
<td>780000 - 195000 (160000 - 400000)</td>
</tr>
<tr>
<td></td>
<td>Roxbury Conglomerate</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td></td>
<td>195000 - 975000 (400000 - 200000)</td>
</tr>
</tbody>
</table>

Table 3.3.1 - Typical Engineering Properties of Foundation Material in Boston [from Johnson, 1989]
<table>
<thead>
<tr>
<th>Soil Stratum Thickness</th>
<th>Typical Profile</th>
<th>South Cove</th>
<th>South Boston ISS4</th>
<th>Transitway</th>
<th>Kneeland St.</th>
<th>P.O. Sq</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill, meters (ft)</td>
<td>3 (10)</td>
<td>9 (30)</td>
<td>2 (7)</td>
<td>7 (24)</td>
<td>9 (28)</td>
<td>4 (14)</td>
</tr>
<tr>
<td>Organics, meters (ft)</td>
<td>2 (5)</td>
<td>9 (30)</td>
<td>0 (15)</td>
<td>5 (12)</td>
<td>4 (19)</td>
<td>6 (0)</td>
</tr>
<tr>
<td>Outwash Deposit, meters (ft)</td>
<td>0 (25)</td>
<td>8 (123)</td>
<td>0 (123)</td>
<td>0 (0)</td>
<td>0 (0)</td>
<td>0 (0)</td>
</tr>
<tr>
<td>Boston Blue Clay, meters (ft)</td>
<td>12 (39)</td>
<td>43 (141)</td>
<td>30 (97)</td>
<td>20 (65)</td>
<td>30 (97)</td>
<td>23 (76)</td>
</tr>
<tr>
<td>Total Depth to bottom of BBC, meters (ft)</td>
<td>31 (103)</td>
<td>152 (499)</td>
<td>39 (127)</td>
<td>70 (229)</td>
<td>82 (268)</td>
<td>66 (217)</td>
</tr>
<tr>
<td>Width of Excavation, B, meters (ft)</td>
<td>21 (70)</td>
<td>61 (200)</td>
<td>34 (110)</td>
<td>10 (32)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B (ft)</td>
<td>3 (10)</td>
<td>4 (12)</td>
<td>3 (10)</td>
<td>4 (13)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strut Spacing, h, meters (ft)</td>
<td>24 (80)</td>
<td>20 (66)</td>
<td>27 (88)</td>
<td>24 (78)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wall Length, L, meters (ft)</td>
<td>15 (50)</td>
<td>12 (40)</td>
<td>18 (58)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.3.2 - Summary of Soil Profiles and Excavation Geometries at five Excavation Project Sites in Boston, MA
<table>
<thead>
<tr>
<th>Changes in OCR</th>
<th>Presence of Cohesionless Layer</th>
<th>Presence of Clay Crust</th>
<th>Moment in the Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. If OCR $&gt; 2$, $\delta_w$ and $\delta_s$ vary linearly with $H$</td>
<td>little influence of $\delta_w$ except for the initial cantilever movement.</td>
<td>negligible impact on initial cantilever movement.</td>
<td>minimal influence on moments</td>
</tr>
<tr>
<td>2. If OCR $&lt; 2$, large reduction in $\delta_w/H$ and $\delta_s/H$ with slight deviation from OCR = 1 (highly non-linear)</td>
<td>distribution of surface settlement is $f(OCR)$</td>
<td>distribution of surface settlement is $f(OCR)$</td>
<td>minimal influence on moments</td>
</tr>
<tr>
<td>3. Wall deflection mode shape is $f(OCR)$</td>
<td>magnitude and distribution of surface settlements are $f(d_s/H)$ for $x &lt; 30$ m</td>
<td>magnitude and distribution of surface settlements are $f(d_s/H)$ for $x &lt; 25$ m and $x &gt; 100$ m from the support wall</td>
<td>minimal influence on moments</td>
</tr>
</tbody>
</table>

Table 3.3.3 - Summary of Results for the Soil Profile Finite Element Experiments
<table>
<thead>
<tr>
<th>Wall Type</th>
<th>Area / Length</th>
<th>(I) / Length</th>
<th>E</th>
<th>(EI)</th>
<th>(EA)</th>
<th>M_all</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>m²/m [in²/ft²]</td>
<td>m⁴/m [in⁴/ft³]</td>
<td>kN/m²</td>
<td>MN-m²/m (EI) / (EI)_{dw, %}</td>
<td>MN/m (EA) / (EA)_{dw, %}</td>
<td>kN·m/m [kips-ft/ft]</td>
</tr>
<tr>
<td>Diaphragm Wall</td>
<td>0.91 [432.0]</td>
<td>6.37E-2 [4665.6]</td>
<td>22.6E6</td>
<td>1440 (100.0)</td>
<td>20570 (100.0)</td>
<td>1200.0 [270.0]</td>
</tr>
<tr>
<td>PZ38 SPW</td>
<td>2.4E-2 [11.18]</td>
<td>3.83E-4 [280.8]</td>
<td>200.0E6</td>
<td>76.6 (5.3)</td>
<td>4800 (22.9)</td>
<td>433.8 [97.5]</td>
</tr>
<tr>
<td>PZ27 SPW</td>
<td>1.7E-2 [7.94]</td>
<td>2.52E-4 [184.2]</td>
<td>200.0E6</td>
<td>50.4 (3.5)</td>
<td>3400 (16.3)</td>
<td>280.0 [62.9]</td>
</tr>
<tr>
<td>PWZ27 SPW</td>
<td>1.7E-2 [7.94]</td>
<td>9.5E-5 [70.1]</td>
<td>200.0E6</td>
<td>19.1 (1.3)</td>
<td>3400 (16.3)</td>
<td>176.2 [39.6]</td>
</tr>
<tr>
<td>PDA27 SPW</td>
<td>1.7E-2 [7.94]</td>
<td>5.44E-5 [39.8]</td>
<td>200.0E6</td>
<td>10.9 (0.76)</td>
<td>3400 (16.3)</td>
<td>99.2 [22.3]</td>
</tr>
<tr>
<td>PMA22 SPW</td>
<td>1.4E-2 [6.47]</td>
<td>2.17E-5 [15.9]</td>
<td>200.0E6</td>
<td>4.3 (0.30)</td>
<td>2800 (13.3)</td>
<td>50.3 [11.3]</td>
</tr>
</tbody>
</table>

Note: SPW = sheet pile wall.
Allowable moment for SPW is based on ASTM A328 steel for SPW; F_b = 25ksi.

Table 3.4.1 - Summary of Support Wall Dimensions and Properties [G1]

<table>
<thead>
<tr>
<th>Analysis Results</th>
<th>Group G1</th>
<th>Group G2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface Settlements and Wall Deflections</td>
<td>Figures 3.4.3 to 3.4.6</td>
<td>Figures 3.4.12 to 3.4.15</td>
</tr>
<tr>
<td>Summary of Maximum Displacements</td>
<td>Figures 3.4.7 to 3.4.8</td>
<td>Figures 3.4.16 to 3.4.20</td>
</tr>
<tr>
<td>Moments in the Wall</td>
<td>Figure 3.4.9</td>
<td>Figures 3.4.21 to 3.4.24</td>
</tr>
<tr>
<td>Strut Loads</td>
<td>Figure 3.4.10</td>
<td>Figures 3.4.25 to 3.4.28</td>
</tr>
</tbody>
</table>

Table 3.4.2 - Summary of Results for Groups G1 and G2 Analyses
<table>
<thead>
<tr>
<th>Wall Type</th>
<th>Final Excavation Depth due to Soil Failure, ( H_s ) (m)</th>
<th>Excavation Depth, ( H_w ), when ( M_{\text{max}} \geq M_{\text{all}} ) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diaphragm Wall</td>
<td>17.5-m</td>
<td>15.0-m</td>
</tr>
<tr>
<td>PZ38 Sheet Pile Wall</td>
<td>12.5-m</td>
<td>( M &lt; M_{\text{all}} )</td>
</tr>
<tr>
<td>PZ27 Sheet Pile Wall</td>
<td>12.5-m</td>
<td>( M &lt; M_{\text{all}} )</td>
</tr>
<tr>
<td>PWZ27 Sheet Pile Wall</td>
<td>12.5-m</td>
<td>12.5-m</td>
</tr>
<tr>
<td>PDA27 Sheet Pile Wall</td>
<td>10.0-m</td>
<td>( M &lt; M_{\text{all}} )</td>
</tr>
<tr>
<td>PMA22 Sheet Pile Wall</td>
<td>10.0-m</td>
<td>10.0-m</td>
</tr>
</tbody>
</table>

Note: see Table 3.4.1 for \( M_{\text{all}} \) for each wall sections

Table 3.4.3 - Summary of Failure Depths for Group G1 Analyses

<table>
<thead>
<tr>
<th>Soil Profile</th>
<th>Concrete Diaphragm Wall</th>
<th>PZ-38 Sheet Pile Wall</th>
<th>PZ-27 Sheet Pile Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>OCR = 1.0</td>
<td>( H_s = 17.5)-m</td>
<td>( H_s = 12.5)-m</td>
<td>( H_s = 12.5)-m</td>
</tr>
<tr>
<td>Profile F1</td>
<td>( H_s = 25.0)-m</td>
<td>( H_s = 20.0)-m</td>
<td>( H_s = 20.0)-m</td>
</tr>
<tr>
<td>Profile F2</td>
<td>( H_s = 25.0)-m</td>
<td>( H_s = 22.5)-m</td>
<td>NA</td>
</tr>
<tr>
<td>Profile F3</td>
<td>( H_s = 22.5)-m</td>
<td>( H_s = 20.0)-m</td>
<td>( H_s = 20.0)-m</td>
</tr>
<tr>
<td>Profile F4</td>
<td>( H_s = 20.0)-m</td>
<td>( H_s = 15.0)-m</td>
<td>NA</td>
</tr>
</tbody>
</table>

Table 3.4.4 - Summary of Final Excavation Depth due to Soil Failure in Group G2 Analyses

<table>
<thead>
<tr>
<th>Soil Profile</th>
<th>Concrete Diaphragm Wall</th>
<th>PZ-38 Sheet Pile Wall</th>
<th>PZ-27 Sheet Pile Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>OCR = 1.0</td>
<td>( H_w = 15.0)-m</td>
<td>( M &lt; M_{\text{all}} )</td>
<td>( M &lt; M_{\text{all}} )</td>
</tr>
<tr>
<td>Profile F1</td>
<td>( M &lt; M_{\text{all}} )</td>
<td>( M &lt; M_{\text{all}} )</td>
<td>( M &lt; M_{\text{all}} )</td>
</tr>
<tr>
<td>Profile F2</td>
<td>( M &lt; M_{\text{all}} )</td>
<td>( M &lt; M_{\text{all}} )</td>
<td>NA</td>
</tr>
<tr>
<td>Profile F3</td>
<td>( H_w = 20.0)-m</td>
<td>( M &lt; M_{\text{all}} )</td>
<td>( M &lt; M_{\text{all}} )</td>
</tr>
<tr>
<td>Profile F4</td>
<td>( H_w = 17.5)-m</td>
<td>( M &lt; M_{\text{all}} )</td>
<td>NA</td>
</tr>
</tbody>
</table>

Note: see Table 3.4.1 for \( M_{\text{all}} \) for each wall sections

Table 3.4.5 - Summary of Final Excavation Depth due to Structural Failure of the Wall for Group G2 Analyses
<table>
<thead>
<tr>
<th>Strut Lateral Spacing, $S$</th>
<th>Strut Stiffness, $k_s$</th>
<th>Design Load per linear length</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0-m</td>
<td>223.1MN/m/m</td>
<td>2230-kN/m [152.4-kips/ft]</td>
</tr>
<tr>
<td>4.0-m</td>
<td>111.5MN/m/m</td>
<td>1115-kN/m [76.2-kips/ft]</td>
</tr>
<tr>
<td>6.0-m</td>
<td>74.4MN/m/m</td>
<td>743-kN/m [50.8-kips/ft]</td>
</tr>
<tr>
<td>8.0-m</td>
<td>55.8MN/m/m</td>
<td>557-kN/m [38.1-kips/ft]</td>
</tr>
<tr>
<td>10.0-m</td>
<td>44.6MN/m/m</td>
<td>446-kN/m [30.5-kips/ft]</td>
</tr>
<tr>
<td>12.0-m</td>
<td>37.2MN/m/m</td>
<td>372-kN/m [25.4-kips/ft]</td>
</tr>
</tbody>
</table>

Note: Steel 76.2-cm in diameter, 1.9-cm thick (30" φ, 0.75" thick) pipe strut is assumed with B/2 = 20.0-m [65.6-ft] and design load of 4460-kN [1000-kips] per strut.

Table 3.4.6 - Summary of Strut Stiffnesses considered in Group H1 Analyses

<table>
<thead>
<tr>
<th>Analysis Results</th>
<th>Group H1</th>
<th>Group H2</th>
<th>Group H3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface Settlements and Wall Deflections</td>
<td>Figures 3.4.31 to 3.4.32</td>
<td>Figures 3.4.38 to 3.4.42</td>
<td>Figures 3.4.47 to 3.4.51</td>
</tr>
<tr>
<td>Summary of Maximum Displacements</td>
<td>Figures 3.4.33 to 3.4.34</td>
<td>Figure 3.4.43</td>
<td>Figure 3.4.52</td>
</tr>
<tr>
<td>Moments in the Wall</td>
<td>Figure 3.4.35</td>
<td>Figure 3.4.44</td>
<td>Figure 3.4.53</td>
</tr>
<tr>
<td>Strut Loads</td>
<td>Figure 3.4.36</td>
<td>Figure 3.4.45</td>
<td>Figure 3.4.54</td>
</tr>
</tbody>
</table>

Table 3.4.7 - Summary of Results for Groups H1, H2, and H3 Analyses
<table>
<thead>
<tr>
<th>Strut Type or Lateral Spacing</th>
<th>Bending Moment in the Wall Group H1 OCR 1, DW</th>
<th>Strut Forces Group H1 OCR 1, DW</th>
<th>Final Excavation Depth Prior to Soil Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rigid Strut</td>
<td>$H_w = 15.0$-m</td>
<td>NA</td>
<td>$H_s = 17.5$-m</td>
</tr>
<tr>
<td>$S = 2.0$-m</td>
<td>$H_w = 15.0$-m</td>
<td>$F_2$ to $F_7 &lt; F_{all}$</td>
<td>$H_s = 17.5$-m</td>
</tr>
<tr>
<td>$S = 4.0$-m</td>
<td>$M &lt; M_{all}$</td>
<td>$F_2$ to $F_6 &lt; F_{all}$</td>
<td>$H_s = 15.0$-m</td>
</tr>
<tr>
<td>$S = 6.0$-m</td>
<td>$M &lt; M_{all}$</td>
<td>$F_2$ to $F_5 &lt; F_{all}$</td>
<td>$H_s = 12.5$-m</td>
</tr>
<tr>
<td>$S = 8.0$-m</td>
<td>$M &lt; M_{all}$</td>
<td>$F_2$ to $F_5 &lt; F_{all}$</td>
<td>$H_s = 12.5$-m</td>
</tr>
<tr>
<td>$S = 10.0$-m</td>
<td>$M &lt; M_{all}$</td>
<td>$H_b = 12.5$ m for $F_4$, $F_2$, $F_3$, &amp; $F_5 &lt; F_{all}$</td>
<td>$H_s = 12.5$-m</td>
</tr>
<tr>
<td>$S = 12.0$-m</td>
<td>$H_w = 12.5$-m</td>
<td>$H_b = 12.5$ m for $F_4$ &amp; $F_5$, $F_2$ &amp; $F_3 &lt; F_{all}$</td>
<td>$H_s = 12.5$-m</td>
</tr>
</tbody>
</table>

Note: $M_{all} (DW) = 1200kN-m/m$; see Table 3.4.6 for $F_{all}$ for each strut spacing

Table 3.4.8 - Summary of Structural Forces for Group H1 Analyses

<table>
<thead>
<tr>
<th>Strut Type or Lateral Spacing</th>
<th>Bending Moment in the Wall Group H2, DW</th>
<th>Strut Forces Group H2 DW</th>
<th>Final Excavation Depth Prior to Soil Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rigid</td>
<td>Real</td>
<td>Rigid</td>
</tr>
<tr>
<td>OCR = 1.0</td>
<td>$H_w = 15.0$-m</td>
<td>$M &lt; M_{all}$</td>
<td>NA</td>
</tr>
<tr>
<td>Profile F1</td>
<td>$M &lt; M_{all}$</td>
<td>$M &lt; M_{all}$</td>
<td>NA</td>
</tr>
<tr>
<td>Profile F2</td>
<td>$M &lt; M_{all}$</td>
<td>$M &lt; M_{all}$</td>
<td>NA</td>
</tr>
<tr>
<td>Profile F3</td>
<td>$H_w = 20.0$-m</td>
<td>$M &lt; M_{all}$</td>
<td>NA</td>
</tr>
<tr>
<td>Profile F4</td>
<td>$H_w = 17.5$-m</td>
<td>$M &lt; M_{all}$</td>
<td>NA</td>
</tr>
</tbody>
</table>

Note: $M_{all} (DW) = 1200kN-m/m$; $F_{all} (S = 6) = 743kN/m$

Table 3.4.9 - Summary of Structural Forces for Group H2 Analyses
<table>
<thead>
<tr>
<th>Strut Type or Lateral Spacing</th>
<th>Bending Moment in the Wall Group H3, SPW</th>
<th>Strut Forces Group H3 SPW</th>
<th>Final Excavation Depth Prior to Soil Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rigid</td>
<td>Real</td>
<td>Rigid</td>
</tr>
<tr>
<td>OCR = 1.0</td>
<td>M &lt; $M_{all}$</td>
<td>M &lt; $M_{all}$</td>
<td>NA</td>
</tr>
<tr>
<td>Profile F1</td>
<td>M &lt; $M_{all}$</td>
<td>M &lt; $M_{all}$</td>
<td>NA</td>
</tr>
<tr>
<td>Profile F2</td>
<td>M &lt; $M_{all}$</td>
<td>M &lt; $M_{all}$</td>
<td>NA</td>
</tr>
<tr>
<td>Profile F3</td>
<td>M &lt; $M_{all}$</td>
<td>M &lt; $M_{all}$</td>
<td>NA</td>
</tr>
<tr>
<td>Profile F4</td>
<td>M &lt; $M_{all}$</td>
<td>M &lt; $M_{all}$</td>
<td>NA</td>
</tr>
</tbody>
</table>

Note: $M_{all}$ (SPW) = 433.8kN·m/m; $F_{all}$ ($S = 6$) = 743.2kN/m

Table 3.4.10 - Summary of Structural Forces for Group H3 Analyses
<table>
<thead>
<tr>
<th>Changes in Support Wall Stiffness</th>
<th>Wall Displacements and Surface Settlements</th>
<th>Strut Loads</th>
<th>Moment in the Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>[Group G]</td>
<td>as EI decreases, δ_w(max) increases and z_{(max)} moves upward toward excavated grade</td>
<td>as EI decreases, strut load also decrease for H &lt; H_s</td>
<td>( IM = f(EI) ), similar distribution for all EI</td>
</tr>
<tr>
<td></td>
<td>as EI decreases δ_v(max) increases and x_{(max)} moves toward the excavation with negligible influence for x &gt; 45m</td>
<td>as EI decreases, strut load increase for H = H_s</td>
<td></td>
</tr>
<tr>
<td></td>
<td>reduction in EI reduces min(L-H) required to remain in bulging mode</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>reduction in EI also yield reduction in failure depth</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \delta_w(max) ) and ( \delta_v(max) ) vary linearly with log(EI)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>effect of increased EI is most pronounced (effective) in soft soil</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Changes in Strut Stiffness</td>
<td>decrease in strut stiffness yields increase in ( \delta_w(max) ) and ( \delta_v(max) ) with negligible impact on settlement at x ≥ 45m.</td>
<td>as strut stiffness decreases, the initial strut force (at installation) decreases yielding a smaller range of fluctuation in forces over the entire construction</td>
<td>as strut stiffness decreases, moment above excavation grade decreases but moment below excavation grade increases</td>
</tr>
<tr>
<td>[Group H]</td>
<td>as strut stiffness decreases, z_{(max)} moves upward with negligible movement in x_{(max)}</td>
<td>for walls with low EI, effects of strut stiffness is less noticeable</td>
<td>location of maximum moment moves upward</td>
</tr>
<tr>
<td></td>
<td>reduction in strut stiffness reduces H_s but does not impact min(L-H) requirement for wall deformation mode</td>
<td></td>
<td>limited effects of low EI walls</td>
</tr>
</tbody>
</table>

Table 3.4.11 - Summary of Results for the Support System Finite Element Experiments
442 Elements, 1413 Nodes, 3292 Degrees of Freedom.

Figure 4.2-2 Finite Element Mesh and Boundary Conditions
Figure 3.1.2 - Initial Conditions and Finite Element Model Excavation Sequence for Hashash's Numerical Study [Hashash, 1992, p.157]
Figure 3.1.3 - Effect of Wall Length on Maximum Wall Bending Moments, Deflections, and Ground Movements (from Hashash and Whittle, 1996)
Figure 3.1.4 - Effect of Wall Length on Lateral Deflections and Surface Settlements for OCR = 1.0 Clay Profile (from Hashash and Whittle, 1996)
Figure 3.1.5 - Effect of Support Spacing on Lateral Wall Deflections and Surface Settlements for OCR = 1.0 Clay Profile (from Hashash and Whittle, 1996)
Figure 3.1.6 - Effect of Support Spacing on Maximum Lateral Wall Deflections and Bending Moments (from Hashash and Whittle, 1996)
Figure 3.1.7 - Comparison of Composite Soil Profile with Measured Data in South Boston (from Hashash and Whittle, 1996)
Figure 3.1.8 - Summary of Excavation Behavior for Overconsolidated Clay Profiles (from Hashash and Whittle, 1996)
Figure 3.1.9 - Comparison of Maximum Wall Deflections and Bending Moments for Composite and Constant OCR Profiles (from Hashash and Whittle, 1996)
Figure 3.1.10 - Estimation of Maximum Lateral Wall Deflections from Numerical Experiments (from Hashash and Whittle, 1996)
(a) Parametric analyses performed by Hashash (1992) to examine effects of wall length (L), strut spacing (h), and clay OCR profile.

(b) Parametric analyses performed in this study to examine effects of wall length (L), depth to bedrock (d_B) and excavation width (B).

Figure 3.2.1 - Scope of parametric study for excavations in clay presented in Sections 3.1 and 3.2
(a) Excavation-induced ground movement measures: wall deflection, surface settlement, surface horizontal displacements, and heave within the excavation

(b) Loads on structural elements supporting the excavation

Figure 3.2.2 - Measures of Ground Movements and Forces on Structural Support System
Note: Negative sign in the wall deflection indicates inward movements toward the excavation.

Figure 3.2.3 - Deflected Wall Shape from Group A Experiments
Figure 3.2.4 - Surface Settlement from Group A Experiments

Maximum Surface Settlement, $\delta v_{(\text{max})}'$ (in)

Surface Settlement, $\delta v$ (in)

Excavation Depth, $H$ (ft)

Maximum Surface Settlement, $\delta v_{(\text{max})}'$ (cm)

Distance from Wall (ft)

Distance from Wall (m)

0.9-m [30-ft] thick concrete diaphragm wall

L = 25.0-m, L = 30.0-m, L = 35.0-m, L = 40.0-m

OCR = 1.0

$B = 80.0$-m = 262.5-ft
$h = 2.5$-m = 8.2-ft
$D = 100.0$-m = 328.1-ft
Figure 3.2.5 - Surface Horizontal Displacement from Group A Experiments
Figure 3.2.6 - Heave within the Excavation from Group A Experiments
Figure 3.2.7a - Strut Loads from Group A Experiments
Figure 3.2.7b - Strut Loads from Group A Experiments

OCR = 1.0
\( d_0 = 100\text{-m} \)
\( B = 80\text{-m} \)
\( h = 2.5\text{-m} \)
Perfect struts

Strut Load [kN/m]

Excavation Depth, H, (m)

Strut Load [kips/ft]

Excavation Depth, H, (ft)
Figure 3.2.8 - Moment Distribution in the wall from Group A Experiments: H = 2.5-m to 12.5-m
Figure 3.2.9 - Moment Distribution in the wall from Group A Experiments: H = 12.5-m to 17.5-m
Figure 3.2.10 - Total Horizontal Stress, Effective Horizontal Stress, and Pore Pressure for Group A Experiments at H = 5.0-m
Figure 3.2.11a - Total Horizontal Stress for Group A Experiments at H = 15.0 m
Figure 3.2.11b - Effective Horizontal Stress for Group A Experiments at $H = 15.0$-m
Figure 3.2.11c - Pore Pressure for Group A Experiments at H = 150-m
GROUP B: (8 cases)

Constant Parameters:
L = 40.0-m [131.2-ft]
d_B = 100.0-m [328.1-ft]
h = 2.5-m [8.2-ft]

Variable:
B = 15.0-m [49.2-ft] (1)
    20.0-m [65.6-ft] (2)
    30.0-m [98.4-ft] (3)
    40.0-m [131.2-ft] (4)
    50.0-m [164.0-ft] (5)
    60.0-m [296.0-ft] (6)
    70.0-m [229.7-ft] (7)
    80.0-m [262.5-ft] (8)

Figure 3.2.12 - Group B Experiments for the Evaluation of the Excavation Width (B) Effects
Figure 3.2.13 - Deflected Wall Shape from Group B Experiments at $H = 17.5$-m
Figure 3.2.14 - Maximum Wall Deflections as Functions of the Excavation Widths From Group B Experiments

Maximum Wall Deflection, \( \delta_{w_{\text{max}}} \), (in)

Excavation Width, B, (ft)

OCR = 1.0
\( D_B = 100.0 \text{m} = 328.1 \text{ft} \)
\( L = 40.0 \text{m} = 131.2 \text{ft} \)
\( h = 2.5 \text{m} = 8.2 \text{ft} \)

0.9 m thick concrete diaphragm wall

17.5 H (m)

Maximum Wall Deflection, \( \delta^{(w_{\text{max}})} \), (cm)
Maximum Surface Settlement, $\delta_{v\text{,}(\text{max})}$ (in)

Excavation Depth, H, (ft)

<table>
<thead>
<tr>
<th>B (m)</th>
<th>$\delta_{v\text{,}(\text{max})}$ (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15.0</td>
<td></td>
</tr>
<tr>
<td>20.0</td>
<td></td>
</tr>
<tr>
<td>30.0</td>
<td></td>
</tr>
<tr>
<td>40.0</td>
<td></td>
</tr>
<tr>
<td>50.0</td>
<td></td>
</tr>
<tr>
<td>60.0</td>
<td></td>
</tr>
<tr>
<td>70.0</td>
<td></td>
</tr>
<tr>
<td>80.0</td>
<td></td>
</tr>
</tbody>
</table>

Surface Settlement, $\delta_v$, (in)

Figure 3.2.15 - Surface Settlement from Group B Experiments at $H = 17.5\text{-m}$. 

$H = 17.5\text{-m}$

$[57.4\text{-ft}]$

$0.9\text{-m (3.0-ft)}$ thick concrete diaphragm wall

$\text{OCR} = 1.0$

$d_s = 1000.0\text{-m} = 328.1\text{-ft}$

$L = 40.0\text{-m} = 131.2\text{-ft}$

$H = 2.5\text{-m} = 8.2\text{-ft}$
Figure 3.2.16 - Maximum Surface Settlement as Functions of Excavation Widths from Group B Experiments.
Figure 3.2.17 - Surface Horizontal Displacement from Group B Experiments at \( H = 17.5 \text{-m} \)
Figure 3.218 - Maximum Surface Horizontal Displacement as Functions of Excavation Widths for Group B Experiments.
Figure 3.2.19 - Heave within the Excavation from Group B Experiments at $H = 17.5$-m

Maximum Heave, $\delta_{hv(\text{max})}$ (in)

Heave within the Excavation, $\delta_{hv}$ (in)

Distance from Center Line (ft) vs. $\delta_{hv}$ (cm)

Distance from Center Line (m) vs. $\delta_{hv}$ (cm)

Heave within the Excavation, $\delta_{hv}$ (cm)

Page 214
Figure 3.2.20 - Maximum Heave within Excavation as Functions of Excavation Widths From Group B Experiments
Figure 3.2.21 - Equivalent Horizontal Pressure Calculated based on Strut Loads from Group B Experiments at H = 17.5-m
Figure 3.2.22 - Moment Distribution in the wall from Group B Experiments at H = 7.5-m and 17.5-m
Figure 3.2.23 - Total Horizontal Stress, Effective Horizontal Stress, and Pore Pressure for Group B Experiments at H = 17.5-m
Figure 3.2.24 - Group C Numerical Experiments for the Evaluation of the Bedrock Depth (d_B) Effects
Figure 3.2.25 - Wall Deflections and Surface Settlements for Group C1 Experiments
Figure 3.2.26a - Wall Deflections and Surface Settlements for Group C2 Experiments
Figure 3.2.26b - Wall Deflections and Surface Settlements for Group C2 Experiments (cont’d)
Figure 3.2.27 - Wall Deflections and Surface Settlements for Group C3 Experiments

Group C3
OCR = 1.0, L = 40.0-m
B = 40.0-m, h = 2.5-m
0.9-m thick concrete diaphragm wall
Figure 3.2.28a - Wall Deflections and Surface Settlements for Group C4 Experiments
Figure 3.2.28b - Wall Deflections and Surface Settlements for Group C4 Experiments (cont’d)
Figure 3.2.29 - Wall Deflections and Surface Settlements for Group C5 Experiments
Figure 3.2.30a - Wall Deflections and Surface Settlements for Group C6 Experiments

Group C6
OCR = 1.0, L = 25.0-m
B = 20.0-m, h = 2.5-m
0.9-m thick concrete diaphragm wall
Figure 3.2.30b - Wall Deflections and Surface Settlements for Group C6 Experiments (cont'd)
Figure 3.2.31a - Normalized Settlement Troughs for Group C Experiments at H = 7.5-m
Figure 3.2.31b - Normalized Settlement Troughs for Group C
Experiments at H = 15.0-m
Figure 3.2.32a - Normalized Maximum Settlements and Maximum Wall Deflections from Group C Experiments
Figure 3.2.32b - Normalized Maximum Settlements and Maximum Wall Deflections from Group C Experiments at $H = 17.5$-m
Figure 3.2.33 - Surface Horizontal Displacement for Group C1 Experiments
Figure 3.2.34 - Surface Horizontal Displacements for Group C2 Experiments
Figure 3.2.35 - Surface Horizontal Displacements for Group C3 Experiments
Figure 3.2.36 - Surface Horizontal Displacements for Group C4 Experiments
GROUP C5
B = 20m
L = 40m

Figure 3.2.37 - Surface Horizontal Displacements for Group C5
Experiments
Figure 3.2.38 - Surface Horizontal Displacements for Group C6 Experiments
Figure 3.2.39 - Heave within Excavation for Group C1 Experiments
Figure 3.2.40 - Heave within Excavation for Group C2 Experiments
Figure 3.2.41 - Heave within Excavation for Group C3 Experiments
Figure 3.2.42 - Heave within Excavation for Group C4 Experiments
Figure 3.2.43 - Heave within Excavation for Group C5 Experiments
Figure 3.2.44 - Heave within Excavation for Group C6 Experiments
Figure 3.2.45a - Strut Loads from Group C1 Experiments

- **Group C1**
- OCR = 1.0
- B = 80 m
- L = 40 m
- h = 2.5 m
- Perfect struts

- Strut 1 (at 0.0 m)
  - $d_B = 100$ m
- Strut 2 (at 2.5 m)
  - $d_B = 75$ m
- Strut 3 (at 5.0 m)
  - $d_B = 50$ m
- Strut 4 (at 7.5 m)

**Excavation Depth, H, (ft)**

**Strut Load [kN / m]**

**Strut Load [kips / ft]**
Figure 3.2.45b - Strut Loads from Group C1 Experiments (cont'd)
Figure 3.2.46a - Strut Loads from Group C2 Experiments
Figure 3.2.46b - Strut Loads from Group C2 Experiments (cont'd)
Figure 3.2.47b - Strut Loads from Group C3 Experiments (cont'd)
Figure 3.2.48 - Moment Distributions in the wall from Group C1 Experiments
Figure 3.2.49 - Moment Distributions in the wall from Group C3 Experiments
Figure 3.2.50 - Total Horizontal Stress, Effective Horizontal Stress, and Pore Pressure for Group C1 at H = 15.0-m
Figure 3.2.51 - Total Horizontal Stress, Effective Horizontal Stress, and Pore Pressure for Group C3 Experiments
GROUP D: Clay Stress History Effects

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>OCR</th>
<th>Undrained Shear Strength, $S_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BBC Properties</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

GROUP E: Overlying Cohesionless Soil Effects

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>OCR</th>
<th>Undrained Shear Strength, $S_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesionless Soil</td>
<td></td>
<td>EP-DP material ($\phi'$, G, $v'$, $\psi$)</td>
</tr>
<tr>
<td>Depth</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BBC Properties</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

GROUP F: Clay Crust Effects

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>OCR</th>
<th>Undrained Shear Strength, $S_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesionless Soil</td>
<td></td>
<td>EP-DP material ($\phi'$, G, $v'$, $\psi$)</td>
</tr>
<tr>
<td>Clay Crust</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.3.1 - Summary of Numerical Analyses used to Evaluate the Impacts of three types of Variations in Soil Profile
GROUP D:  
(16 cases)

Constant Parameters:

\[
\begin{align*}
B &= 80.0\text{-m} \quad [262.5\text{-ft}] \\
d_B &= 100.0\text{-m} \quad [328.1\text{-ft}] \\
h &= 2.5\text{-m} \quad [8.2\text{-ft}]
\end{align*}
\]

Variables:

\[
\begin{align*}
L &= 25.0\text{-m} \quad [82.0\text{-ft}] \\
    &\quad 30.0\text{-m} \quad [98.4\text{-ft}] \\
    &\quad 35.0\text{-m} \quad [114.8\text{-ft}] \\
    &\quad 40.0\text{-m} \quad [131.2\text{-ft}] \\
OCR &= 1.0, 1.15, 1.25, 1.7, \text{ and } 2.0
\end{align*}
\]

Figure 3.3.2 - Specification of Group D Experiments for Evaluating the Effects of Clay Stress History
Figure 3.3.3 - Wall Deflections for Group D Experiments with OCR = 1.15 Clay
Figure 3.3.5 - Wall Deflections for Group D Experiments with OCR = 1.25 Clay
Figure 3.3.6 - Surface Settlements for Group D Experiments with OCR = 1.25 Clay
Figure 3.3.7 - Wall Deflections for Group D Experiments with OCR = 1.70 Clay
Figure 3.3.9 - Wall Deflections for Group D Experiments with OCR = 2.0 Clay
Figure 3.10 - Surface Settlements for Group D Experiments with OCR = 2.0 Clay
Figure 3.3.11 - Normalized Maximum Wall Deflections and Surface Settlements for Group D Experiments
Figure 3.3.12 - Normalized Deflected Wall Shapes for Group D Experiments at H = 2.5, 5.0, 12.5, and 17.5-m
Figure 3.3.13a - Normalized Surface Settlements for Group D Experiments at H = 2.5-m and 12.5-m
Maximum Surface Settlement, $\delta_{v_{(max)}}$ (in)

Excavation Depth, $H$, (ft)

OCR 2
OCR 1.7
OCR 1.25
OCR 1.15
OCR 1

$L = 40.0\,m$
$B = 80.0\,m$
$d_b = 100.0\,m$

Maximum Surface Settlement, $\delta_{v_{(max)}}$ (cm)

Distance from Wall (ft)

$B = 80.0\,m$
$L = 40.0\,m$
$d_b = 2.5\,m$

Distance from Wall (m)

Figure 3.3.13b - Normalized Surface Settlements for Group D Experiments at $H = 5.0\,m$ and $17.5\,m$
Figure 3.3.14 - Moment Distributions in the Wall for Group D Experiments at H = 17.5-m
Figure 3.3.15 - Equivalent Horizontal Stress Calculated from Strut Loads for Group D Experiments at H = 17.5-m
Figure 3.3.16a - Typical Soil Profile in the Downtown Boston Area
[Johnson, 1989]

Figure 3.3.16b - Idealized Soil Profiles for the Numerical Experiments in Group E
GROUP E: (4 cases)

Constant Parameters:
- $B = 40.0$-m [131.2-ft]
- $d_B = 50.0$-m [164.0-ft]
- $L = 25.0$-m [82.0-ft]
- $h = 2.5$-m [8.2-ft]

Variables:

<table>
<thead>
<tr>
<th>Clay Stress History Profile:</th>
<th>Thickness of Cohesionless Soil Stratum, $d_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5.0-m [16.4-ft]</td>
</tr>
<tr>
<td>OCR = 1.0</td>
<td>Profile E1</td>
</tr>
<tr>
<td>OCR = 1.15</td>
<td>Profile E3</td>
</tr>
</tbody>
</table>

Figure 3.3.17 - Group E Numerical Experiment for the Evaluation of the Overlying Cohesionless Soil Effects
Figure 3.3.18 - Wall Deflections from Group E Experiments With Cohesionless Soil Overlying OCR = 1 Clay
Figure 3.3.20 - Wall Deflections for Group E Experiments with Cohesionles Soil Overlying OCR = 1.15 Clay
Figure 3.3.21 - Surface Settlements for Group E Experiments with Cohesionless Soil, Overlying OCR = 1.15 Clay

Page 279
<table>
<thead>
<tr>
<th>$H = 5.0\text{-m}$</th>
<th>OCR $= 1.0$</th>
<th>$H = 12.5\text{-m}$</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1.png" alt="Figure 3.3.22" /></td>
<td><img src="image2.png" alt="Figure 3.3.22" /></td>
<td><img src="image3.png" alt="Figure 3.3.22" /></td>
</tr>
<tr>
<td>$(a)$</td>
<td>$(b)$</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$H = 15.0\text{-m}$</th>
<th>$H = 17.5\text{-m}$</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image4.png" alt="Figure 3.3.22" /></td>
<td><img src="image5.png" alt="Figure 3.3.22" /></td>
</tr>
<tr>
<td>$\delta_v(\max)$</td>
<td>contour interval: 0.5-cm</td>
</tr>
<tr>
<td>$u_y = \delta_v(\max)$</td>
<td></td>
</tr>
<tr>
<td>$(c)$</td>
<td>$(d)$</td>
</tr>
</tbody>
</table>

Figure 3.3.22 - Contours of Vertical Displacements, $u_y$, for an Excavation with $B = 40.0\text{-m}$, $L = 25.0\text{-m}$, $d_B = 50.0\text{-m}$, $h = 2.5\text{-m}$, and OCR $= 1.0$ at Excavation Depths of 5.0-m, 12.5-m, 15.0-m, and 17.5-m.
Figure 3.3.23 - Contours of Vertical Displacements, \( u_y \), for an Excavation with \( B = 40.0 \text{-m}, L = 25.0 \text{-m}, d_B = 50.0 \text{-m}, h = 2.5 \text{-m}, \) and Profile E1 at Excavation Depths of 5.0-m, 12.5-m, 15.0-m, and 17.5-m.
Figure 3.3.24 - Contours of Vertical Displacements, \( u_y \), for an Excavation with \( B = 40.0 \text{-m}, L = 25.0 \text{-m}, d = 50.0 \text{-m}, h = 2.5 \text{-m}, \) and Profile E2 at Excavation Depths of 5.0-m, 12.5-m, 15.0-m, and 17.5-m.
Figure 3.3.25 - Contours of Vertical Displacements, $u_y$, for an Excavation with $B = 40.0$-m, $L = 25.0$-m, $d_B = 50.0$-m, $h = 2.5$-m, and OCR = 1.15 at Excavation Depths of 5.0-m, 12.5-m, 15.0-m, and 17.5-m
Figure 3.3.26 - Contours of Vertical Displacements, \( u_y \), for an Excavation with \( B = 40.0\)-m, \( L = 25.0\)-m, \( d_B = 50.0\)-m, \( h = 2.5\)-m, and Profile E3 at Excavation Depths of 5.0-m, 12.5-m, 15.0-m, and 17.5-m
Figure 3.3.27 - Contours of Vertical Displacements, $u_y$, for an Excavation with $B = 40.0$-m, $L = 25.0$-m, $d_B = 50.0$-m, $h = 2.5$-m, and Profile E4 at Excavation Depths of 5.0-m, 12.5-m, 15.0-m, and 17.5-m
Figure 3.3.28 - Moment Distributions in the Wall for Group E Experiments with Underlying OCR = 1.0 Clay (Profiles E1 & E2)
Figure 3.3.29 - Moment Distributions in the Wall for Group E Experiments with Underlying OCR = 1.15 Clay (Profiles E3 and E4)
Figure 3.3.30 - Strut Loads for Group E Experiments with Underlying OCR = 1.0 Clay (Profiles E1 and E2)
Figure 3.3.31 - Strut Loads for Group E Experiments with Underlying OCR = 1.15 Clay (Profiles E3 and E4)
Figure 3.3.32 - Stress History Profile measured at the South Boston Special Test Site using 1-D consolidation Tests
Figure 3.3.33 - Undrained Shear Strength Profile Measured at the South Boston Special Test Site
Figure 3.3.34 - Stress History Profiles (F1, F2, F3 and F4) Included in Group F Numerical Analysis: Clay Crust Effects
GROUP F: (4 cases)

**Constant Parameters:**
- \( B = 40.0 \text{-m} \) [131.2-ft]
- \( d_B = 50.0 \text{-m} \) [164.0-ft]
- \( L = 25.0 \text{-m} \) [82.0-ft]
- \( h = 2.5 \text{-m} \) [8.2-ft]

**Variable: Soil Profile**

<table>
<thead>
<tr>
<th>Soil Profile</th>
<th>Profile F1</th>
<th>Profile F2</th>
<th>Profile F3</th>
<th>Profile F4</th>
</tr>
</thead>
<tbody>
<tr>
<td>( d_s )</td>
<td>15.0-m</td>
<td>15.0-m</td>
<td>15.0-m</td>
<td>5.0-m</td>
</tr>
<tr>
<td>Clay Crust Depth</td>
<td>15 to 27.5m</td>
<td>15 to 27.5m</td>
<td>15 to 27.5m</td>
<td>5 to 17.5m</td>
</tr>
<tr>
<td>Main Clay Stratum</td>
<td>OCR=1</td>
<td>OCR=1.15</td>
<td>OCR=1</td>
<td>OCR=1</td>
</tr>
</tbody>
</table>

Figure 3.3.35 - Group F Numerical Experiments for the Evaluation of the Clay Crust Effects
Figure 3.3.36 - Wall Deflections for Group F Experiments with Soil Profiles F1 and F2
Figure 3.3.37 - Surface Settlements from Group F Experiments with Soil Profiles F1 and F2
Figure 3.3.38 - Wall Deflections from Group F Experiments with Soil Profiles F1 and F3
Figure 3.3.9 - Surface Settlements from Group F Experiments with Soil Profiles F1 and F3
Figure 3.3.40 - Wall Deflections for Group F Experiments with Soil Profiles F3 and F4
Figure 3.3.41 - Surface Settlements from Group F Experiments with Soil Profiles F3 and F4

Maximum Surface Settlement, $\delta_{v(max)}$ (in)

Surface Settlement, $\delta_v$ (in)

Undrained Analysis
Effects of Clay Crust
$L = 250.0$ m
$B = 400.0$ m
$d_b = 50.0$ m

Distance from Wall (ft)
Distance from Wall (m)

Page 299
Figure 3.3.42 - Moment Distribution in the Wall for Group F Experiments with Soil Profiles F1, F2, F3, and F4
Figure 3.3.43 - Strut Loads for Group F Experiments with Soil Profiles F1, F2, F3, and F4
GROUP G: (16 Cases)

Constant Parameters:

\[ \begin{align*}
B &= 40.0 \text{-m} \quad [131.2 \text{-ft}] \\
\frac{B}{2} &= 20 \text{-m} \quad [65.6 \text{-ft}] \\
h &= 2.5 \text{-m} \quad [8.2 \text{-ft}] \\
L &= 25.0 \text{-m} \quad [82.0 \text{-ft}] \\
d_B &= 50.0 \text{-m} \quad [164.0 \text{-ft}]
\end{align*} \]

Variables:

Support Wall Type: [G1]
0.9-m Concrete Diaphragm Wall
PZ-38, PZ-27, PWZ-27, PDA-27, and PMA-22 Sheetpile Sections

Soil Profiles: [G2]
OCR = 1.0, Profile F1, Profile F2, Profile F3, and Profile F4

Figure 3.4.1 - Group G Numerical Experiment for the Evaluation of the Support Structures
GROUP G1:

Constant Parameters:
- \( B = 40.0\text{-m} \) [131.2-ft]
- \( h = 2.5\text{-m} \) [8.2-ft]
- OCR = 1.0 Soil Profile
- \( d_B = 50.0\text{-m} \) [164.0-ft]
- \( L = 25.0\text{-m} \) [82.0-ft]
- Rigid Supports

Variables:
- Support Wall Type:
  - 0.9-m Concrete Diaphragm Wall: 1442.0
  - PZ-38: 76.7
  - PZ-27: 50.3
  - PWZ-27: 19.1
  - PDA-27: 10.9
  - PMA-22: 4.3

(6 Analyses)

Figure 3.4.2 - Group G1 Numerical Experiments for the Evaluation of the Effects of Support Wall Stiffness
Figure 3.4.3 - Wall Deflections and Surface Settlements for Group G1
Experiments at H = 5.0-m
Figure 3.4.4 - Wall Deflections and Surface Settlements for Group G1 Experiments at H = 7.5-m
Figure 3.4.5 - Wall Deflections and Surface Settlements for Group G1
Experiments at $H = 10.0$-m
Figure 3.4.6 - Wall Deflections and Surface Settlements for Group G1
Experiments at H = 12.5-m
Figure 3.4.7 - Summary of Maximum Displacements as Function of Excavation Depth for Group G1 Analyses
Figure 3.4.8 - Summary of Maximum Displacements as Function of the Bending Stiffness of the Support Wall for Group G1 Analyses
Figure 3.4.9 - Moment Distribution in the Wall for Group G1 Experiments at H = 10m
Figure 3.4.10 - Summary of Strut Forces for Group G1 Analyses, OCR = 1.0, L = 25-m
B = 40-m, $d_B = 50$-m, and perfect struts at $h = 2.5$-m
GROUP G2:

Constant Parameters:

<table>
<thead>
<tr>
<th>B</th>
<th>40.0-m</th>
<th>[131.2-ft]</th>
</tr>
</thead>
<tbody>
<tr>
<td>h</td>
<td>2.5-m</td>
<td>[8.2-ft]</td>
</tr>
<tr>
<td></td>
<td>Rigid Supports</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>d_B</th>
<th>50.0-m</th>
<th>[164.0-ft]</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
<td>25.0-m</td>
<td>[82.0-ft]</td>
</tr>
</tbody>
</table>

Variables:
Support Wall Type and Soil Profile:

<table>
<thead>
<tr>
<th>Support Wall Type</th>
<th>F1</th>
<th>F2</th>
<th>F3</th>
<th>F4</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.9-m Concrete DW</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>[EI]=1442.0MN-m²/m</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PZ-38</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>[EI]=76.7MN-m²/m</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PZ-27</td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>[EI]=50.3MN-m²/m</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.4.11 - Group G2 Numerical Experiments for the Evaluation of the Combined Effects of Support Wall Stiffness and Soil Profile
Figure 3.4.12 - Wall Deflections and Surface Settlements for Group G2 Experiments with Soil Profile F1
Figure 3.4.13 - Wall Deflections and Surface Settlements for Group G2
Experiments with Soil Profile F2
Figure 3.4.14 - Wall Deflections and Surface Settlements for Group G2 Experiments with Soil Profile F3
Figure 3.4.15 - Wall Deflections and Surface Settlements for Group G2 Experiments with Soil Profile F4
Figure 3.4.16 - Summary of Maximum Displacements as Functions of Excavation Depth for Group G2 Experiments with Profile F1
Figure 3.4.17 - Summary of Maximum Displacements as Functions of Excavation Depth for Group G2 Experiments with Profile F2
Figure 3.4.18 - Summary of Maximum Displacements as Functions of Excavation Depth for Group G2 Experiments with Profile F3
Figure 3.4.19 - Summary of Maximum Displacements as Functions of Excavation Depth for Group G2 Experiments with Profile F4
Figure 3.4.20 - Summary of Maximum Displacements as Functions of the Bending Stiffness of the Support Wall for Group G2 Experiments
Figure 3.4.21 - Moment Distribution in the Wall for Group G2 Experiments with Soil Profile F1 at $H = 10.0$-m [32.8-ft]
Figure 3.4.22 - Moment Distribution in the Wall for Group G2 Experiments with Soil Profile F2 at H = 10.0-m [32.8-ft]
Figure 3.4.23 - Moment Distribution in the Wall for Group G2 Experiments with Profile F3 at H = 10.0-m [32.8-ft]
Figure 3.4.24 - Moment Distribution in the Wall for Group G2 Experiments with Soil Profile F4 at H = 10.0-m [32.8-ft]
Figure 3.4.25b - Summary of Strut Forces for Group G2 Experiments with Profile F1 (cont'd)
Figure 3.426a - Summary of Strut Forces for Group G2 Experiments with Profile F2
Excavation Depth (ft)

Group G2
Profile F2

- 0.9-m DW
- PZ-38 SPW

Strut Load (kN/m)

Strut 5 (at 10.0-m)
Strut 6 (at 12.5-m)
Strut 7 (at 15.0-m)
Strut 8 (at 17.5-m)

Figure 3.4.26b - Summary of Strut Forces for Group G2 Experiments with Profile F2 (cont'd)
Figure 3.4.27a - Summary of Strut Forces for Group G2 Experiments with Profile F3
Figure 3.4.27b - Summary of Strut Forces for Group G2 Experiments with Profile F3 (cont'd)
Figure 3.4.28 - Summary of Strut Forces for Group G2 Experiments with Profile F4
Figure 3.4.28b - Summary of Strut Forces for Group G2 Experiments with Profile F4 (cont'd)
GROUP H: (25 Cases)

Constant Parameters:

\[ \begin{align*}
B &= 40.0\text{-m} [131.2\text{-ft}] \\
h &= 2.5\text{-m} [8.2\text{-ft}] \\
d_B &= 50.0\text{-m} [164.0\text{-ft}] \\
L &= 25.0\text{-m} [82.0\text{-ft}] 
\end{align*} \]

Variables:

- Strut Stiffness:
  - Perfect Struts vs. Real Struts with different lateral spacing, S [H1]

- Soil Profiles:
  - OCR = 1.0, Profile F1, Profile F2, Profile F3, and Profile F4

- Support Wall:
  - 0.9-m Concrete Diaphragm Wall [H2]
  - PZ-38 Sheetpile Wall [H3]

<table>
<thead>
<tr>
<th>Profile</th>
<th>DW [H1] (S = 2, 4, 6, 8, 10, &amp; 12-m)</th>
<th>DW [H2] (S = 6-m)</th>
<th>SPW [H3] (S = 6-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OCR 1.0</td>
<td>6X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Profile F1</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Profile F2</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Profile F3</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Profile F4</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

Figure 3.4.29 - Group H Numerical Experiments for the Evaluation of the Strut Stiffness
GROUP H1: Strut Stiffness  

**Constant Parameters:**
- \( B = 40.0 \text{-m} \) [131.2-ft]
- \( h = 2.5 \text{-m} \) [8.2-ft]
- OCR = 1.0 Soil Profile
- \( d_B = 50.0 \text{-m} \) [164.0-ft]
- \( L = 25.0 \text{-m} \) [82.0-ft]
- 0.9-m thick diaphragm wall

**Variables:**
- Lateral Strut Spacing, \( S \)
  - 2.0-m
  - 4.0-m
  - 6.0-m
  - 8.0-m
  - 10.0-m
  - 12.0-m
- \( k \) [MN/m/m] [kips/ft/ft]
  - 223.1
  - 111.5
  - 74.4
  - 55.8
  - 44.6
  - 37.2
  - 4647.5
  - 2323.7
  - 1549.2
  - 1161.9
  - 929.5
  - 774.58

*Note: for experiments with real struts, level 1 struts are omitted*

Figure 3.4.30 - Group H1 Numerical Experiments for the Evaluation of the Effects of Wall Support Stiffness
Figure 3.4.31 - Wall Deflections and Surface Settlements for Group H1 Experiments, H = 5-m, 7.5-m, and 10-m
Figure 3.4.32 - Wall Deflections and Surface Settlements for Group H1 Experiments at H = 12.5, 15, and 17.5-m
Figure 3.4.33 - Summary of Maximum Displacements as Functions of Excavation Depth for Group H1 Experiments
Figure 3.4.34 - Summary of Maximum Displacements as Functions of the Wall Support Stiffness (k) for Group H1 Experiments
Figure 3.4.35 - Moment Distribution in the Wall for Group H1 Experiments
Figure 3.4.36b - Summary of Strut Forces for Group H1 Experiments (cont'd)
**GROUP H2:**  
(10 Cases)

*Constant Parameters:*

- \( B = 40.0 \text{-m} \) [131.2-ft] \( d_B = 50.0 \text{-m} \) [164.0-ft]
- \( h = 2.5 \text{-m} \) [8.2-ft] \( L = 25.0 \text{-m} \) [82.0-ft]  
0.9-m thick concrete diaphragm wall

*Variables:*

Strut Stiffness:
- Perfect Struts vs. Real Struts with lateral spacing of 6.0-m, \( S = 6.0 \text{-m} = 19.7 \text{-ft} \)

Soil Profiles:
- OCR = 1.0, Profile F1, Profile F2, Profile F3, and Profile F4

---

Figure 3.4.37 - Group H2 Numerical Experiments for the Evaluation of the Strut Stiffness (DW with rigid vs. real struts)
Figure 3.4.38 - Wall Deflections and Surface Settlements for Group H2 Experiments with Constant OCR = 1 Clay Profile
Figure 3.4.39 - Wall Deflections and Surface Settlements for Group H2 Experiments with Profile F1
Figure 3.4.40 - Wall Deflections and Surface Settlements for Group H2 Experiments with Profile F2
Figure 3.4.41 - Wall Deflections and Surface Settlements for Group H2 Experiments with Profile F3
Figure 3.4.42 - Wall Deflections and Surface Settlements for Group H2 Experiments with Profile F4
Figure 3.4.43 - Summary of Maximum Displacements as Function of Excavation Depth for Group H2 Experiments
Figure 3.4.44 - Moment Distribution in the Wall for Group H2 Experiments
Figure 3.4.45a - Summary of Strut Forces for Group H2 Experiments (Struts 1 to 4)
Figure 3.4.45b - Summary of Strut Forces for Group H2 Experiments (Struts 5 to 8)
Perfect Struts or
76-cm φ, 1.9-cm thick steel pipe struts with S = 6.0-m

b/2 = 20-m [65.6-ft]
L = 25.0-m [82.0-ft]

PZ-38 Sheet Pile Wall

Clay Stratum
OCR = 1.0, Profile F1, Profile F2, Profile F3, and Profile F4
BBC Properties

GROUP H3: (10 Cases)

Constant Parameters:

\[ B = 40.0\text{-m} \quad [131.2\text{-ft}] \]
\[ h = 2.5\text{-m} \quad [8.2\text{-ft}] \]
\[ L = 25.0\text{-m} \quad [82.0\text{-ft}] \]
\[ d_B = 50.0\text{-m} \quad [164.0\text{-ft}] \]

Variables:
Strut Stiffness:
Perfect Struts vs. Real Struts with lateral spacing of 6.0-m, S = 6.0-m = 19.7-ft

Soil Profiles:
OCR = 1.0, Profile F1, Profile F2, Profile F3, and Profile F4

Figure 3.4.46 - Group H3 Numerical Experiments for the Evaluation of the Strut Stiffness (SPW with rigid vs. real struts)
Figure 3.4.47 - Wall Deflections and Surface Settlements for Group H3 Experiments with OCR = 1 Clay
Figure 3.4.48 - Wall Deflections and Surface Settlements for Group H3 Experiments with Profile F1
Figure 3.4.49 - Wall Deflections and Surface Settlements for Group H3 Experiments with Profile F2
Figure 3.4.50 - Wall Deflections and Surface Settlements for Group H3 Experiments with Profile F3
Figure 3.4.51 - Wall Deflections and Surface Settlements for Group H3 Experiments with Profile F4
Figure 3.4.52 - Summary of Maximum Displacements as Functions of Excavation Depth for Group H3 Experiments
Figure 3.4.53 - Moment Distribution in the Wall for Group H3 Experiments
Figure 3.4.54b - Strut Forces for Group H3 Experiments (Struts 5 to 8)
CHAPTER 4

Analyses of Three Deep Excavations in Boston

The previous chapter, Chapter 3, used numerical experiments on idealized excavations in clay-dominated soil profiles in order to assess the impact of various factors such as excavation geometry, soil profile, and support systems on the predicted ground movements and structural loads. The goal of this chapter is to evaluate the predictive capabilities and limitations of the numerical analyses through detailed case studies of three deep excavation projects in the Boston area. Figure 4.1.1 shows the location of these three projects: (1) MBTA South Cove Station (completed in 1971), (2) South Boston (Central Artery Section 4A, completed in 1995), and (3) MBTA Transitway (currently under design). All three sites have subsurface profiles with Boston Blue Clay as the dominant soil stratum.

The analyses of the South Cove and South Boston projects are assessed through direct comparison with measured data. The third case study, the MBTA Transitway, is currently in the preliminary design stage; thus the numerical analyses were performed in order to assist initial design decisions for the proposed lateral earth support, predicted soil movements, and construction sequence. Description of the three sites and the corresponding
analyses are presented in Sections 4.1, 4.2, and 4.3. Evaluation of the overall predictive capabilities and limitations of the numerical analyses is summarized in Section 4.4.

4.1 SOUTH COVE CASE STUDY

The first case study focuses on the measured performance of excavations for the New England Medical Center\textsuperscript{69} Subway Station (on the MBTA Orange Line) in Boston, Massachusetts [Lambe et al., 1972, Jaworski, 1973, Russell, 1993]. Although this project was completed more than 20 years ago, it remains one of the most comprehensively documented excavations in the Boston area. This site also possesses three features which were evaluated in the parametric analyses presented in Chapter 3: 1) the site stratigraphy includes a relatively deep deposit (25.0-m) of Boston Blue Clay (BBC); 2) the structural design includes adjacent sections (with similar cross-sections, excavation history, and soil profile) supported by slurry walls and sheet pile walls (i.e. large difference in bending stiffness, see Section 3.4; and 3) the toe of the walls are embedded in soft normally consolidated BBC, and do not extend into the underlying till and bedrock.

The site description, soil profile, available field instruments, construction sequence, and numerical analyses results are presented in the following sections.

4.1.1 South Cove Site Description

The South Cove project, better known as the New England Medical Center Station on the Orange line, is located south of downtown Boston (see Figure 4.1.1). The construction for this subway station took place during the late

\textsuperscript{69} This site is also referred to as the "South Cove" project.
1960's and early 1970's. Figure 4.1.2 shows a partial plan of the South Cove site and elevations corresponding to the final excavation grade. The station itself has an overall length of 183.0-m, average excavation depth of 15.0-m, and ranges in width from 18.0 to 24.0-m. In order to protect the seven story Don Bosco School building from excavation-induced ground movements, a 60.0-m long section of excavation was supported by a 0.9-m thick, 24.4-m deep concrete diaphragm wall (Stations 111+80 through 113+86; Figure 4.1.2), with three levels of cross-lot bracing (see Figure 4.1.3). The remainder of the excavation, also supported by three levels of cross-lot bracing\textsuperscript{70}, uses sheet pile walls (section BZ350, Arbed Columeta, Belgium Steel) driven to approximately the same depth as the diaphragm wall (see Figures 4.1.2 and 4.1.4).

The analyses of the South Cove site focuses on two sections: (i) diaphragm wall Section A-A (Station 113+40, Figures 4.1.2 and 4.1.3), which is in close proximity (less than 1.5-m) to the north-east corner of the Don Bosco school building and contains extensive field instrumentation to monitor ground movements and pore pressures; and (ii) sheet pile wall Section B-B (Station 111+40, Figures 4.1.2 and 4.1.4), where inclinometer SI-11 measures the lateral movements of the sheet pile wall.

The soil profile, location of field instruments, and construction sequence are described in Sections 4.1.2 to 4.1.4. The numerical model, which is based on the existing soil condition, structural supports, and excavation sequence, is described in Section 4.1.5. Sections 4.1.6 and 4.1.7 contain results of the numerical analyses for Sections A-A and B-B. Summaries and concluding remarks regarding this site are presented in Section 4.1.8.

\textsuperscript{70} Figure 4.1.2 contains one serious error: the level E struts were NOT installed at Section B-B.
4.1.2 Soil Profile

The soil profile for Section A-A (Station 113+40) is based mainly on data from boring BH-10 which is located approximately 5.0-m to the north (see Figure 4.1.2). Figure 4.2.3 shows the subsurface profile for Section A-A which consists of four major soil layers: 2.1-m of fill (mainly cider ash) and 4.6-m of hard yellow clay overlying the main deposit of Boston Blue Clay which is further sub-divided by consistency into very stiff to stiff (at depths less than 18.3-m, and stiff to medium clay layers (below depth of 18.3-m). The clay is underlain by glacial till (mainly sand and gravel with very little clay) at a depth of 31.7-m. The ground water table is located at a depth of 5.2-m with hydrostatic in-situ pore water pressures ($u_0$) in the underlying clay. There is also a perched water table within the fill at a depth of 1.5-m.

The soil profile at Section B-B (Station 111+40) is based mainly on information from Boring BH-9 (see Figure 4.1.2). The subsurface soil profile is very similar to conditions at Section A-A and also comprises of four major soil layers (see Figure 4.1.4): 2.1-m of fill (mainly fine to coarse sand) and 4.0-m of hard yellow clay overlying the main 25.9-m thick deposit of Boston Blue Clay. The clay is underlain by glacial till at a depth of 32.0-m. The ground surface at Section B-B is approximately 1.4-m lower than at Section A-A; nevertheless, the groundwater table is at the same elevation.

The original site investigation was very limited in scope, with no data for the fill and almost no information on the properties of the upper yellow clay (unit weights of these materials were estimated by inspection). The original test program consisted of Atterberg limits, Torvane index strengths, laboratory permeability and 1-D consolidation tests on block samples of the Boston Blue Clay, together with field vane strength data (obtained by MIT and
J.P. Collins & Associates\textsuperscript{71}. Figure 4.1.5 summarizes the physical and engineering properties from the site investigation including the index properties \((w, w_L, w_p)\), undrained field vane (and torvane) shear strength, \(s_{uFV}\), and the pre-consolidation pressure, \(\sigma'_{p}\), as reported by Jaworski [1973].

4.1.3 Field Instruments

The South Cove project was the first project to use a cast in-situ reinforced concrete diaphragm wall for an excavation in Boston Blue Clay. The design was selected explicitly to minimize ground movements and prevent damage to the adjacent Don Bosco school building. Hence, this site was relatively well instrumented in the vicinity of Section A-A (see Figure 4.1.2). The field instruments at Section A-A consist of the following:

i.) Lateral deflections are measured by two Wilson slope indicators (SI-4 and SI-1, Figures 4.1.2 and 4.1.3). The casing for SI-4 is cast within the west diaphragm wall and extends only to the base of the wall (depth 24.4-m), while SI-1 is located approximately 1.5-m behind the wall and extends to a depth of approximately 30.0-m (within the lower BBC). Hence, neither instrument has a well defined basal datum\textsuperscript{72}.

ii.) Soil deformations within the excavation were monitored by a series of heave rods (V, Figure 4.1.2), while surface movements of the surrounding ground were surveyed by a series of level points (LP) and settlement screws.

iii) The changes in strut loads were measured by Telemac type F-2 vibrating wire strain gauges attached to each of the three strut levels. However, some of these data are unreliable as the instruments were not properly calibrated in the field (Russell, 1993).

\textsuperscript{71} J.P. Collins & Associates also performed CIU and UU triaxial tests on 3" Shelby tube samples. However, these data are considered less reliable than the field vane measurements.\textsuperscript{72} The top of the casing is used as the reference point.
iv.) Genor vibrating wire piezometers were installed before the diaphragm wall was constructed; and pore water pressures were monitored continuously throughout the excavation by these piezometers located at various depths within the clay: P1 - P3 inside the excavation and P4-P7 adjacent to the Don Bosco Building.

In addition to monitoring the performance of the cast in-situ concrete diaphragm wall, another main focus of the field instrumentation program for the South Cove site was to compare the performance of diaphragm wall (Section A-A) with that of sheet pile wall (Section B-B). The available data at Section B-B consist of the following:

i.) Lateral deflections are measured by one Wilson slope indicator (SI-11). The casing for SI-11 is cast within the west sheet pile wall and extends to the base of the wall (depth 24.0-m). Similar to slope indicators located within Section A-A, this instrument lack a point of fixed reference, therefore, all lateral deflections were computed relative to the measured movements of the top of the casing.

ii.) Pore water pressures were monitored by four Casagrande-type hydraulic piezometers (PH-4, PH-5, PH-6, and PH-7) located at a depths of 13.5-m and at a distance from 3.1-m to 48.4-m from the west wall.

4.1.4 Construction Sequence

The construction sequence at Sections A-A and B-B follow the similar excavation sequence consisting of removal of soil followed by the installation of struts. The major difference between these two section is that struts at Section A-A were preloaded whereas struts at Section B-B were not preloaded.
Detailed descriptions of the construction schedule and activities at these two sections are provided in Sections 4.1.4.1 and 4.1.4.2.

4.1.4.1 Section A-A

The slurry wall was installed during the period of January to April 1969. Excavation began at Section A-A (Station 113+40) on April 29, 1969, which is referred as Construction Day 0 (CD 0) in the Section A-A analyses, and was completed on October 31, 1969 (i.e., CD 184). CD 184 also marks the beginning of mudmat installation and subsequent construction of the station structure. Figure 4.1.6 represents the excavation sequence in a series of 8 discrete steps based on the construction activities reported by Jaworski [1973]. The following points should be noted regarding activities at Section A-A:

1. According to the field records, the excavation took place concurrently along the east and west walls and hence, the excavated ground profile remained relatively symmetric throughout the construction period. The subsequent numerical analyses only consider one half of the cross-section (the half-width, B/2 = 10.7-m).

2. There were three levels of cross-lot bracing, referred to as levels B, C, and D (14WF127, 14WF103, and 14WF176) located at depths of 3.7-m, 9.1-m, and 12.2-m respectively (see Figure 4.1.3). Each tier of strut consisted of steel H-section columns spaced at 3.7-m centers (see Figure 4.1.2). Jaworski [1973] provided detailed documentation of the installation schedule as well as the measured pre-load forces in each strut (nominally these are equal to 50% of the design lateral forces). The magnitudes of the pre-load at Section A-A are averaged from the data reported for struts #44, 45, and 46 (see Figure 4.1.2).

3. The initial phase (Steps 2 to 4, Figure 4.1.6) involved the excavation of a central access ramp to a maximum depth of 8.7-m, leaving a construction berm for installing the first level of
bracing (level B, Step 473). Steps 6 and 7 show similar excavated profiles (but different berm sizes) prior to installation of the level C and level D bracing, respectively. The exact widths and sideslopes of the construction berms were not recorded. The subsequent analyses assume a constant berm width of 2.4-m and a 1:1 slope, which is probably steeper than the actual construction (and hence, the analyses will tend to underestimate the stabilizing effects of the berm).

Figure 4.1.7 shows the comparison of the construction schedule incorporated in the numerical model and the actual progress documented by Jaworski (1973).

4.1.4.2 Section B-B

The excavation at Section B-B is supported by a sheet pile wall (Section BZ 350, Arbed Columeta, Belgium Steel) which was installed on June 7, 1969. Excavation began at Section B-B on June 6, 1969 (COD 0) and was completed on August 15, 1969 (COD 71) ready for mudmat installation and subsequent construction of the station structure. The excavation grade elevations during the construction period are shown in Figure 4.1.8, while Figure 4.1.9 presents the idealized construction sequence in a series of 11 discrete steps74 based on the record of site activities reported by Jaworski [1973]. The following points should be noted for activities at Section B-B:

1. Similar to Section A-A, excavation along the east and west walls of Section B-B also progressed concurrently; hence, the excavation ground profile remained symmetric at all times. Consequently, the numerical analyses for Section B-B also

73 The small access trench in step 2 is used to install the wale beam for level B struts.
74 Note that the initial geostatic condition, Step 1, is not shown in the figure.
considers one half of the cross-section (with half-width, B/2, 9.8-
m).

2. The sheetpile walls at Section B-B consist of three tiers of steel H-
section columns (12WF45, 14WF103, and 12WF119) spaced at
approximately 4.5-m centers at depths 1.7-m, 6.7-m, and 11.0-m.
Unlike Section A-A, the struts at Section B-B were NOT
preloaded; they were simply wedged in place with wood blocks.

3. The construction sequence at Section B-B included excavating a
central access ramp, installation of struts (at Steps 6, 8, and 10)
and the removal of the berms adjacent to the wall (Steps 5 and
9). The exact widths and sideslopes of the construction berms
were not recorded. The analyses for Section B-B assume a
constant berm width of 2.4-m and a 1:1 slope.

4.1.5 Finite Element Model and Input Parameters

The soil input parameters and the finite element models for the South
Cove analyses are described in this section.

4.1.5.1 In Situ Conditions

The in-situ vertical effective stress, \( \sigma'_{v0} \), used in the analysis is based on
the unit weights quoted by Jaworski [1973], based on empirical correlations,
and the measured pore water pressure conditions, \( u_0 \), (see Figure 4.1.5). For
the top fill layer, no laboratory tests were performed; all the soil properties
were extrapolated based on the blow counts from the standard penetration
tests.

The properties of the underlying clay layer are based on field vane tests
as well as a number of laboratory tests:
- 5 consolidated isotropic undrained triaxial tests (CIU)
- 5 unconsolidated undrained triaxial tests (UU)
- Torvane shear tests
- 11 consolidation tests
- 5 permeability tests
- 13 sets of Atterberg Limits

The available measured data are rather limited for the assessment of the engineering properties of the soils especially at depths less than 10.0-m.

Based on the measured field vane strengths and the oedometer test results from the original South Cove site investigation, two pre-consolidation pressure profiles are estimated: Profile A and Profile B. The pre-consolidation pressure Profile A (see Figure 4.1.5) is obtained by assuming that the field vane strengths are the most reliable data. The preconsolidation pressure is calculated by matching the measured strength ratio $s_{uFV}/\sigma'_{v0}$ with the computed value of $S_u^{DSS}/\sigma'_{vc}$ vs. OCR. The resulting pre-consolidation pressure Profile A lies within the range of measured values of maximum past pressure in BBC as reported by Jaworski [1973]. The second profile, Profile C, is obtained by re-evaluating the original oedometer test data from the South Cove site. Profile C represents the best estimate of the pre-consolidation pressures within the Boston Blue Clay layer and concluded that the values of $\sigma'_{p}$ quoted by Jaworski [1973] underestimate significantly the true pre-consolidation pressures measured in the laboratory tests.

In general, the two profiles, Profiles A and C, consist of a clay crust (at depths 6.5-m to 11.0-m) overlying normally to lightly overconsolidated BBC (below a depth of 11.0-m). Both profiles assume the same state for the stiff yellow clay. This stiff yellow clay is highly overconsolidated (OCR= 8.0 to 20.0) and lie outside of the recommended range for the use of the MIT-E3 soil model [OCR ≤ 8, Whittle, 1987, 1993]; consequently, unrealistically high values of $K_0$ (2.0 to 3.5) for this stiff yellow clay layer are predicted by the MIT-E3 soil model. To address this concern, a third profile, Profile D, is
introduced. Profile D is identical to Profile C below a depth of 10.0-m [32.8-ft]; however, within the stiff yellow clay stratum, more realistic values of \( K_0 \) (1.2 to 1.7), which are based on recent laboratory test data [Sheahan, 1991], are used.

A simple parametric study was performed to assess the impact of the uncertainty in the stress history profile as well as the uncertainties in the lateral earth pressure. Similar to results presented in Section 3.3, the results of this simple study suggest that the uncertainties in the pre-consolidation pressures of the lightly overconsolidated BBC layer (Profile A vs. C) are of secondary importance in predicting the performance of the South Cove excavations; however, the uncertainties in the \( K_0 \) stress conditions (Profile C vs. D) have a major influence on the mode shape and magnitudes of wall and ground movements for shallow excavations (less than 10.0-m deep). Since the assumed \( K_0 \) values in Profiles A and C (as predicted by MIT-E3) are unrealistically high at high OCR's, Profile D represents the most reliable combination of state parameters \( \sigma' \) and \( K_0 \) and is used in the analyses of Section A-A and Section B-B.

4.1.5.2 Finite Element Model

Figure 4.1.10 shows the finite element mesh and boundary conditions used to simulate the excavation sequence for Section A-A described above (a similar mesh was used for Section B-B but is not shown here). In general, the models assume plane strain conditions with no displacements at the base of the mesh which corresponds to the clay/glacial deposit interface (\( d_B = 31.7 \text{-m} \)). The construction records suggest that the cross-sections remain approximately symmetric throughout the construction period, and hence, the analyses simulated on-half of the section. The model assumes that no lateral
displacements or pore pressure change occur at the far right hand boundary. Pore pressures at the till interface are also assumed to remain constant\textsuperscript{75}.

Three types of elements are used within the FE models: 1) 8-node (biquadratic displacement) solid isoparametric elements for modeling structural wall and building foundation; 2) 1-D spring elements modeling the cross-lot bracing; and 3) 8-node mixed quadrilateral element with eight displacement nodes (biquadratic displacement) and four pore pressure nodes (bilinear pore pressure) for the soil.

Both finite element models assume that the walls are initially wished-in-place with a unit weight equal to that of the surrounding soil. The 0.9-m thick concrete diaphragm walls are modeled as an elastic solid\textsuperscript{76} with $E = 2.3 \times 10^4$ MPa and $\nu = 0.15$. The sheetpile walls at Section 3-B is also modeled using an elastic solid element with an equivalent wall thickness of 0.42-m, Young's modulus of $7.9 \times 10^3$ MPa and $\nu = 0.1$. These equivalent values are selected to replicate the axial and bending stiffness of the sheetpile wall section\textsuperscript{77} [after Day & Potts, 1993].

The effects of the nearby Don Bosco school building on predictions of excavation performance are difficult to assess. Based on site documentation, the building has a floating foundation and therefore, it is reasonable to assume that the initial geostatic stresses in the ground are not affected significantly by the presence of the foundation. However, the distribution of the ground movements caused by the excavation will clearly be affected by the

\textsuperscript{75} This assumption implies that the till has a much higher permeability than the clay and acts as a confined aquifer. Unfortunately, no tests were performed in the till stratum, however, observations at other sites in the Boston area [P.O. Square, Whittle et. al, 1994 and South Boston, Whelan, 1995] suggest this assumption to be valid at this site.

\textsuperscript{76} Plane strain, 8-node isoparametric solid element with elastic material properties.

\textsuperscript{77}Equivalent axial stiffness: $E_A = E_f A_f$
Equivalent bending stiffness: $E_I = E_f I_f$
where $E_A, A_f$, and $I_f$ are the actual Young's modulus, cross-sectional area and second moment of area for the sheetpile section.
rigidity of the foundation. Unfortunately, the three dimensional footprint of the building cannot be replicated in a 2-D planar finite element model. Though its shear properties will depend on the actual geometry and the materials used in the structure [cf. Burland & Wroth, 1974] as a first approximation, the analyses treat the foundation as a solid concrete base with the same elastic properties as the diaphragm wall\(^7\).

The three level of struts are represented by 1-D spring elements with elastic properties. The axial stiffness for these elastic springs are obtained by the following relationship:

\[
\frac{k}{L} = \frac{E_s A_s}{(B/2)S}
\]

where:
- \( k \) is the spring constant [kN/m]
- \( L \) is the unit length of the wall [m]
- \( E_s \) is the Young’s modulus of steel [200-GN/m\(^2\)]
- \( A_s \) is the cross sectional area of the steel strut [Section A-A: 236, 200, and 334-cm\(^2\) for levels B, C, and D; Section B-B: 85.5, 195.5 and 225.8-cm\(^2\) for levels B, C, and D]
- \( B/2 \) is the half-length of the cross-lot strut
  - [10.7-m for Section A-A and 9.75-m for Section B-B]
- \( S \) is the average spacing of the strut
  - [3.66-m for Section A-A and 4.45-m for Section B-B]

For Section A-A, the pre-load forces are based on the field measurements [Jaworski, 1973] for struts #44, 45, and 46. Table 4.1.1 summarizes the preload and strut stiffness properties. Figure 4.1.11 illustrates the numerical

---

\(^7\) A simple numerical experiment was performed to assess the impact of including the foundation in the model. The results indicate that the foundation rigidity reduces the differential settlements beneath the structure. It also reduces the maximum surface settlement by approximately 0.3-cm at all stages of excavation. Though this magnitude represents a large proportion of settlement during early stages of excavation, it is only 10% of the predicted settlement at final stages of excavation (stage 7/8). The foundation also reduces the maximum wall deflections by 0.35-cm to 0.5-cm but has little impact on the deflected shape of the wall.
procedures used to model the installation and pre-loading of the cross-lot bracing. In the case of Section B-B, there are no pre-load forces and the analyses assume prefect contact (i.e. no gap) between the sheetpile wall and the bracing once the struts are installed. This assumption is unlikely to be valid in the field given that the struts were simply wedged in place by wood blocks. However, there is insufficient data to refine the calculations since the size of the gap and the stiffness of the wood blocks were not documented or studied.

Two soil models are used to describe the behavior of the fill and clay. The effective stress-strain-strength properties of the fill are described by an elasto-plastic model with a Drucker-Prager failure criterion with a non-associated flow rule (zero dilation at failure). The analyses use the same fill input parameters selected by Whittle et al. [1993]: earth pressure coefficient, $K_0 = 0.5$, elastic shear stiffness, $G/\sigma'_v = 25$, Poisson's ratio, $\nu' = 0.3$, and friction angle in plane strain compression, $\phi'_{PS} = 30^\circ$. The constitutive behavior of the hard yellow and blue clay layers are represented by MIT-E3 [Whittle, 1987; Whittle and Kavvadas, 1996]—using material input parameters previously selected for BBC (see Table 3.1.1). This assumption of a fixed set of material input parameters implies that all points within the clay profile will exhibit the same normalized engineering properties. The shear stiffness and undrained shear strength of the clay are then controlled by the in-situ effective stress state ($K_0 = \sigma'_h / \sigma'_v$) and the pre-consolidation pressure, $\sigma'_p$ ($OCR = \sigma'_p / \sigma'_v$). Soil Profile D, presented in the previous section, is considered to provide the best representation of BBC stress history profile at this site; therefore, this profile is assumed in analyses at both Section A-A and Section B-B.
4.1.6 Numerical Results for Section A-A

This section compares the observed measurements with the predictions of wall deflections, surface settlements, and pore pressures obtained from two finite element analyses using MIT-E3 soil model with soil Profile D assuming plane strain conditions. Results from undrained analysis as well as partially drained analysis (which incorporates the actual construction time and permeability of the soil) are presented. In both cases, the idealized 8-stage excavation schedule is assumed in the FE model (see Figure 4.1.6).

The real time analyses assume a water table in the retained soil at 5.2-m below ground level and hydrostatic in-situ pore pressures at the base of the underlying clay (top of till). Since there are no records of dewatering at the site, the partially drained analysis also assume that the water table within the excavation coincides with the excavation grade. The partial drainage of BBC is controlled by a constant permeability, \( k = 5.0 \times 10^{-8} \text{ cm/sec} = 1.42 \times 10^{-4} \text{ ft/day} \). The resulting wall deflections, ground surface settlements, pore pressures, and strut loads obtained from these two analyses are summarized in the following paragraphs.

Figures 4.1.12 through 4.1.15 compare the predictions of the lateral wall deflections and surface settlements from undrained and partially drained analyses with field measurements at Stages 2, 6, 7, and 8.

At Stage 2 (Figure 4.1.12), the partial drainage causes a small increase in the initial cantilever movement of the wall but also reduces the surface settlements. Consequently, the measured data are in better agreement with the partially drained than the undrained analysis in terms of both settlement and wall deflection.

At subsequent stages of excavation (Stages 6 to 8, Figures 4.1.13 to 4.1.15), the partially drained analysis predicts maximum wall movements which are
approximately 35% less than the corresponding undrained calculations, and maximum surface settlements which are 50% smaller. The measured surface settlements at Stages 6, 7, and 8 are in much better agreement with the partially drained analysis. However, the scatter in the inclinometer data precludes any definite conclusion on the wall deformations. The predicted wall deflection obtained from the partially drained analysis lies between the range of the measured data (SI-4 and SI-1), but shows higher bending in the diaphragm wall than either of the inclinometers. The difference between the predicted and the measured behavior at depths less than 10.0-m are likely to be caused by the uncertainties in $K_0$ conditions and the stress-strain-strength behavior\(^7\) in the upper BBC and yellow clay.

Figure 4.1.16 compares the predictions of pore pressures from the undrained and partially drained analyses with the measured data. There are six vibrating wire piezometers (P-1 through P-7\(^3\)) and two hydraulic piezometers (P-1-H and PH-8) installed in the BBC at Section A-A. The predicted pore pressure from the undrained and the partially drained analyses reported at the locations of the piezometers are shown in Figure 4.1.16 along with the measured data. As expected, the assumed drainage condition has the greatest effect on the predicted pore pressures near the excavation (piezometers P-1, P-1-H, P-2, P-4, P-5, and P-6). The impact of the drainage condition is less visible at distances greater than B ($x > 22.3$-m) from the excavation. In all cases, the measured data followed the same trends as the predictions. Piezometers P-1, P-1-H, P-5, P-6, and PH-8 all show excellent agreement with the predicted pore pressures. In the cases of P-2, P-4, and P-7,

\(^7\) The clay within this region is highly overconsolidated and is beyond the recommended range for the MIT-E3 soil model.
\(^3\) An additional piezometer, P-3, located in the underlying till, is not shown in Figure 4.1.16. The piezometer showed negligible changes during excavation and was damaged in September, 1969.
there is a constant offset between the measured and predicted values. One possible cause for this difference is incorrect zero reading of the instruments.

The measured and predicted strut loads for levels B, C, and D are compared in Figure 4.1.17. As discussed previously, the actual measured pre-loads are used directly in the FE analyses; therefore, the measured and the predicted strut loads are initially equal. However, the measured strut loads during subsequent construction stages are only 40% to 50% of those predicted by the finite element analyses. It is likely that this discrepancy is caused by the poor quality of the field instrumentation\textsuperscript{81}.

Overall, the predictions from partially drained analysis of Section A-A give reasonable agreement with measured wall deflections, surface settlements, and pore pressures. These results suggest that the FE model is able to capture the main aspects of the deformation response of the soil at the South Cove site as well as the structural properties (wall and pre-stressed struts).

4.1.7 Numerical Results for Section B-B

This section compares the observed measurements with the predictions of wall deflections and pore pressures obtained from two finite element analyses\textsuperscript{82} using MIT-E3 soil model with soil Profile D for Section B-B assuming undrained\textsuperscript{83} and plane strain conditions. Figure 4.1.18 through 4.1.22 compare the predictions of these analyses with the field measurements from slope indicator SI-11 at Steps 5, 6, 8, 9, and 11.

\textsuperscript{81} According to Jaworski [1973, p. 46]: "The initial gauges on struts B-31 through B-40 were incorrectly installed. Telemac type SB-90 strain gauges were installed on these struts when the error was discovered."
\textsuperscript{82} One analysis includes all three levels of struts (B, C, and D); the other analysis only include two levels of struts (B and D).
\textsuperscript{83} A partially drained analysis was not performed at Section B-B.
At Step 5 (Figure 4.1.18), the measured wall deflection (recorded on July 9, 1969) is at approximately the same magnitude as the predicted movements; however, the mode shape of the two wall deflections differ significantly -- the measured data indicate the presence of a support while the first level of strut was not installed until one day later (Step 6 - July 10, 1969). Consequently, the same set of measured wall movements is also compared with the predicted wall movements at Step 6 (Figure 4.1.19). The same mode of wall deflection as predicted was observed in the field; however, the actual movements were only about half the magnitude of the predicted deformations. The difference can be attributed to three possible factors:

1. The analyses assume an additional 1.83-m of excavated material between Steps 5 and 6. The wall deflection measurements may have been taken after Step 5 but prior to the excavation reaching a depth of 6.7-m (end of Step 6). Since the original documents did not include the exact excavation configuration at the time of measurement, this factor cannot be verified.

2. The top of the inclinometer casing is the reference point for computing wall deflections. Small errors in the surveyed position of the top of the casing are possible.

3. Including partial drainage in the analysis will reduce the predicted wall deflections at Stages 5 and 6 (see results for Section A-A, Figure 4.1.12)

All three factors cited above can account for the difference in the measured and the predicted wall movements at Stages 5 and 6.

From Step 8 onwards (Figures 4.1.20 to 4.1.22), the FE predictions greatly underestimate the magnitude of the maximum wall movements and
describe a deflected wall shape which differs significantly from the inclinometer data. The measured data at Step 8 show \( \delta_{w_{\text{max}}} = 7.9\text{-cm} \) [3.1-in] occurring at the top of berm elevation, and zero lateral displacement at the toe of the wall. In contrast, the predictions show \( \delta_w = 2.9\text{-cm} \) [1.1-in] at the berm elevation, decreasing to \( \delta_w = 0.8\text{-cm} \) [0.3-in] at the toe of the wall. Further studies have been performed to evaluate the source of these large discrepancies between predicted and measured behavior. Three possible explanations are as follows:

1. The analysis greatly overestimates the axial stiffness of the cross-lot bracing. The FE analyses assume that the full axial stiffness of the strut (steel H-column sections) become effective immediately after strut installation. In contrast, the struts were actually wedged against the sheet pile wall using wooden blocks. Hence the actual stiffness of the bracing may be controlled by compression of the wooden blocks. In order to investigate this issue, a second analysis was performed with the Level C bracing completely removed. Figures 4.1.20 through 4.1.22 show results of this analysis. The results show little difference in the predicted wall movements while the berm remains in place (Step 8; Figure 4.1.20). However, once the berm is removed at Step 9, maximum wall movements increase from \( \delta_{w_{\text{max}}} = 3.6\text{-cm} \) to 5.1-cm (with and without Strut C; Figure 4.1.21). The stiffness of Strut C has negligible influence on predicted to movements of the sheet pile wall. The measured wall movements, therefore, cannot be explained only by the stiffness of Strut C.

2. Bending movements within the sheetpile wall can be estimated from the measured lateral deflections at Step 8 (assuming that the wall
remains elastic). These calculations show a maximum bending movement \( M_{\text{max}} = 250 \, \text{kN-m/m} \), which is very close to the yield moment of the sheetpile wall section (bending about a centroidal axis in the cross-sectional plane of the excavation at Section B-B). Thus, the measured data imply that the wall has either yielded or is very close to its yield point. The exact yield moment of the wall is not known, since there were additional deflections of the wall in the longitudinal direction\(^{84}\) (i.e. wall displacements in the North-South direction, Figure 4.1.2).

In order to investigate the effects of wall yielding, a series of FE analyses were performed with specified yield moments for the wall. Figure 4.1.20 shows results for the analysis with \( M_y = 25 \, \text{kN-m/m} \) (i.e., approximately 10% of the expected yield moment for bending in the B-B plane). These results show a small increase in wall deflection at the elevation of the soil berm in Step 8, but no effect on the movements of the wall at depth. Hence, yielding of the wall (even at reduced values of \( M_y \)) does not give a consistent explanation of the reported wall deflections in Steps 8 to 11.

3. Similar to the analyses at Section A-A, the behaviors of the highly overconsolidated stiff yellow clay and the upper layer of BBC are likely to influence the deformations obtained from the numerical analyses. Unfortunately, there is insufficient data regarding these soil properties to justify additional analyses with different input parameters. Nevertheless, the uncertainties in these soil parameters will only

---

\(^{84}\) Russell [1993] recalls large longitudinal wall movements (North-South direction) parallel to the excavation. Inclinometer SI-11 moved about 3.7-cm southward, while SI-12 (7.3-m further along the wall) moved 7.5-cm northward. These data suggest a bow-shaped deflection of the wall at Section B-B (i.e. combined bending about x-x and y-y axis) which would cause further reduction of the section yield moment.
impact movements within the top 10.0-m and still cannot explain the observed wall deflection below the excavation grade.

Despite of the three possible explanations described above, there are no alternative explanations which are consistent with the measured data. Limitations or errors in the inclinometer data appear to be the most likely cause since it is highly unlikely that the toe of a 24.1-m long wall will show zero lateral displacement for a 15.0-m deep excavation especially when the wall is embedded within normally to lightly overconsolidated BBC.

Figure 4.1.23 compares the measured and predicted pore pressure heads, $H_p$, from the two undrained analyses (one with and one without Strut C). At a distance greater than B (one excavation width) from the excavation (i.e. PH-6 and PH-7), the predicted pore pressures are very similar to the measured data (with $H_p$ decreasing by several meters). For the locations closer to the wall, the predictions show much larger decreases in the pore pressures than are measured by piezometers PH-4 or PH-5. These differences may be due to partial drainage (flow) through the semi-permeable sheet pile wall itself.

4.1.8 Summary of Findings from the South Cove Study

Two typical cross-sections were selected from the South Cove project for detailed numerical analyses: 1) Section A-A, diaphragm wall supported by three levels of pre-stressed cross-lot bracing; and 2) B-B, sheet pile wall section supported by three levels of bracing with zero prestress.

Reliable predictions of lateral wall deflections, surface settlements, and pore pressures have been achieved for Section A-A using a real-time (i.e. including partial drainage) finite element analysis, together with the MIT-E3 soil model and carefully defined initial stress states and stress history profiles.
in the BBC. The presence of an adjacent foundation (Don Bosco School) affects the measured surface settlements and has also been included in the analyses. Inconsistencies in the measured lateral wall deflections (by inclinometers within the wall and in the retained soil) reflect limitations of the measured data and preclude more definitive conclusions. The uncertainties in the stress history profile mainly impacts the lateral movements within the top 10-m of the wall.

The analyses of Section B-B have not achieved satisfactory agreement with measured lateral wall deflections. The measured data show much larger wall deflections than were observed at Section A-A. According to the analyses, these differences are not due to the bending stiffness of the wall itself, but are more directly related to differences in the pre-load used in the cross-lot bracing systems\(^{85}\) as well as the uncertainty in the soil behavior \(^{86}\) of the stiff yellow clay. Further analyses have investigated the effects of support stiffness and wall yielding, but they have not found a consistent explanation for the measured wall movements and suggest there may be significant errors in the data.

Overall, this case study demonstrated that although field data were well recorded and correlated to construction activities at the South Cove site, the limited site investigation program and the inconsistent field measurements pose potential difficulties in the evaluation of the excavation using finite element analysis. This case study also demonstrated that, despite of these potential difficulties, the numerical technique developed thus far can produce reliable predictions of behaviors of excavations in clay supported by

\(^{85}\) At Section A-A, the struts are prestressed where as struts at Section B-B have zero pre-stress and are wedged in place with a wooden block.

\(^{86}\) This layer is highly overconsolidated (OCR between 8 and 20) which is beyond the recommended range of the MIT-E3 model.
diaphragm wall; however, this level of performance has not been duplicated for excavations in clay supported by sheetpile walls.
4.2 DEEP EXCAVATION IN SOUTH BOSTON

The second case study focuses on the excavations for a depressed roadway in South Boston corresponding to Section 4A of the Central Artery/Tunnel Project (CA/T) (see Figure 4.1.1). Figure 4.2.1 presents a plan of the entire 4A contract including the relative locations of adjacent buildings and field instruments. The selected section for analysis, ISS4 on Figure 4.2.1, involves a 12.2-m deep and 61.0-m wide excavation in 31.0-m of surficial deposits consisting of fill, organic deposits, and Boston Blue Clay. The excavation was supported by a 16.8-m deep sheet pile wall on the south side and a 32-m deep concrete diaphragm wall keyed into the underlying bedrock on the north side. Both walls were further reinforced by three tiers of tiebacks. The tiebacks on the south side (sheet pile wall) were anchored in the clay crust whereas the tiebacks located on the north side (diaphragm wall) were grouted in the bedrock.

Detailed documentation of the construction activities, field measurements, and other pertinent information for this site can be found in Whelan [1995]. Brief summaries of the site description, soil profile, field instruments, and the construction sequence are abstracted from Whelan [1995] and presented in Sections 4.2.1 to 4.2.4, respectively. These descriptions of site conditions and construction activities are the basis for the finite element model described in Section 4.2.5. Sections 4.2.6 and 4.2.8 present the results of two finite element models simulating the excavation at this site, referred to as Base case and the Revised analyses, respectively. The Base analysis, which represent the initial model for this site, is presented in Section 4.2.6. Evaluation of the Base Case analysis is presented in Section 4.2.7. The Revised analysis, covered in Section 4.2.8, includes a series of modifications based on the interpretation of the predictions for the Base Case
model. The predictive capabilities of the numerical models are evaluated based on the comparisons between the numerical results and the actual field measurements. The overall performance of the numerical simulations for this South Boston Site is assessed in Section 4.2.9.

4.2.1 Site Description

Figure 4.2.1 shows the plan view of the excavation in South Boston with the positions of field instruments and eight Instrumented Sections (ISS-1 to ISS-8). Based on the types field instruments, excavation geometry, support system, and available documented construction activities, Whelan [1995] selected ISS-4 for detailed study; and this ISS4 section is also the focus of this numerical study.

Section ISS-4 is located adjacent to Building A covering a distance of approximately 30.5-m along the roadway alignment. Figures 4.2.2 and 4.2.3 show the plan view as well as the cross-sectional view at ISS-4. The north side of the excavation (along Building A) is supported by a 0.9-m thick reinforced concrete diaphragm wall spanning between Stations 76+00 and 77+60. This concrete diaphragm wall extends 0.6-m into the underlying argillitic bedrock\(^{87}\). Table 4.2.1 summarizes the dimensions of the tiebacks and the locations of the tieback anchors. On the North wall, the three tiers of tiebacks have a vertical inclination of 45° and are grouted in the underlying glacial till and bedrock.

The south side of ISS-4 is supported by Arbed AZ-18 steel sheetpile wall extending 16.8-m below the surface. Properties of the AZ-18 sheet piling are summarized in Figure 4.2.4. There are also three tiers of tiebacks located on the South wall at depths 2.4-m, 6.1-m, and 9.8-m (Table 4.2.1). The top two

\(^{87}\) Bedrock is at a depth of 31.4-m.
tiers are inclined at 22° from the horizontal and the bottom tier has an inclination angle of 20° (see Figure 4.2.3). These tiebacks are grouted in the clay crust (i.e. at depths 11.5 to 16.8-m) of the Boston Blue Clay stratum. The south wall tieback dimensions are also included in Table 4.2.1.

Each tieback along the North and South walls was subjected to and required to pass a proof test\textsuperscript{88}. These proof tests provided load-elongation data for each of the tiebacks thus measured the "effective"\textsuperscript{89} moduli for these tiebacks. These proof tests also assure that each tieback is capable with withstanding 150% of the design load\textsuperscript{90}. Table 4.2.2 summarizes the effective moduli, $E$, for each tier of the tiebacks along both walls as well as the lock-off loads. As in other published case histories, the field measured $E$ is lower than the theoretical modulus of the steel, and can be partially attributed to unwinding of the strand wires during loading [Xanthakos, 1991]. A general relationship between the theoretical value and field-measurements is difficult to obtain as the unwinding of strand wires is highly dependent on the restraining capacity of the stressing system.

4.2.2 Subsurface Conditions

The subsurface soil conditions at this site are based on an extensive site investigation program which included 62 borings with in-situ and laboratory tests as well as data from different projects in the vicinity. The selected average soil profile for ISS-4, shown on Figure 4.2.3, was defined based on borings near this section and installation logs for the inclinometers and the

\textsuperscript{88} If the tieback fails the initial proof test, the tieback is regrunted and then retested. This process is repeated until the tieback passes the proof test.

\textsuperscript{89} The load-elongation data capture the stretching of the tendon as well as the grouted zone; therefore, the resulting moduli are referred to as the "effective" moduli.

\textsuperscript{90} The contractor selected 170 tons as the failure for the tiebacks along the south wall. The design load was defined as 1/2 of the failure load (equaling 85 tons).
piezometers. This average soil profile consists of 31.4-m of surficial deposits founded on glacial deposits and bedrock. The surficial deposits include four strata: 2.6-m of granular fill, 4.6-m of cohesive fill, 4.6-m of organic deposit, and 19.7-m of Boston Blue Clay. The engineering properties of each stratum are summarized in Table 4.2.3. Detailed descriptions of these soil strata can be found in Whelan [1995] and Haley & Aldrich, Inc. site investigation reports [1991a, 1991b, and 1993].

As discussed in Section 3.3, the stress history profile within the clay stratum has a major impact on the prediction of excavation-induced soil movements especially for lightly overconsolidated clay layers. Although large variations in stress history and strength were reported along the alignment of CA/T Section 4A, the selected stress history and undrained shear strength profiles presented in Figures 4.2.5a and 4.2.5b are representative for ISS-4 [Whelan, 1995]. These profiles were obtained by re-evaluation of the original consolidation test data on the BBC samples [Ladd, 1994]. In general, this stress history profile is rather typical in the Boston area with a deep clay crust overlying a nearly normally consolidated clay. Detailed descriptions of the clay properties at this site can be found in Whelan [1995].

In terms of the groundwater conditions, the Boston Blue Clay layer has a much lower hydraulic conductivity than the overlying or underlying strata. The upper aquifer consists of granular fill, cohesive fill, and organic deposits with an initial groundwater table at a depth of 1.2-m. The underlying glacial deposits and the weathered bedrock define a lower aquifer system whose initial piezometric elevation is 30.5-m (i.e. at depth of 3.0-m). Within the BBC stratum, the initial piezometric elevation is assumed to vary linearly

---

91 The ground surface is at Elevation 110.0-ft; therefore, the initial piezometric level in the upper aquifer is El. 106.0-ft, while the lower aquifer is at El. 100.0-ft. The project datum for the CA/T project is 100-ft below the National Geodetic Vertical Datum of 1929 (NGVD 1929)
with depth from elevation of 32.3-m at the top of the clay layer (depth 11.8-m) to elevation of 30.5-m at the bottom of the clay layer (depth 31.4-m). Figure 4.2.6 shows the initial pore pressure as well as the total and effective vertical stresses at ISS-4.

4.2.3 Field Instruments

A number of geotechnical instruments were installed at ISS-4 in order to measure the soil movements and pore pressures in the vicinity of the excavation (see Figures 4.2.2 and 4.2.3, and Table 4.2.4). The horizontal movements are monitored by two inclinometers (INC-101 and INC-102\textsuperscript{92}) and one inclinometer/probe extensometer (IPE-113) located near the two support walls. Surface settlements are measured at nine monitoring points (DMP4's and DMP2's\textsuperscript{93}). Heave measurements are reported at the four multi-point heave gauges (MPHG) and the inclinometer/probe extensometer (IPE-113). Pore pressures are measured at 15 locations including two observation wells (OW-002 and OW-16), one open standpipe piezometer (OSPZ-106), and 12 vibration wire piezometers (VWPZ's). With the exception of the two inclinometers (INC-101 and INC-102) at the two walls where the initial readings were taken during the first stage of excavation, the remaining instruments were installed and initialized prior to any construction activities at ISS-4.

\textsuperscript{92} INC-102 is located inside the North diaphragm wall while INC-101 is placed approximately 0.6-m behind the South sheetpile wall.

\textsuperscript{93} DMP2's are deflection monitoring points placed on existing structures whereas DMP4's are placed on "natural" ground surface.
4.2.4 Construction Activities

Whelan [1995] reconstructed the excavation history at ISS-4 through careful examination of field engineers' reports and construction photographs. Based on this information, Whelan was able to document the complete construction history at ISS-4 which included the duration of excavation steps, dates of tieback installation and lock-off, and pumping activities along the excavation alignment.

Figures 4.2.7a through 4.2.7h illustrate the eight major excavation steps at ISS-4 described by Whelan. Figure 4.2.7i shows the overall construction schedule along the North and South walls and the Center portion of the proposed roadway. Excavation of the first tier (Step 1, Figure 4.2.7a) commenced around March 22, 1993. The removal of soil, followed by the installation and lock-off of the tiebacks, continued for the next several months (Steps 2 to 5, Figures 4.2.7b - 4.2.7e). By mid-September, 1993, after excavation of five lifts to a depth of 9.1-m, installation of tiedowns within the excavation progressed in conjunction with the excavation of the last two tiers (Steps 6 & 7, Figures 4.2.7f and 4.2.7g). Final invert for this section was poured by late-May, 1994.

In order to ensure a factor of safety against hydrostatic uplift of equal or greater than 1.2 at all stages of the excavation (MHD, 1992), dewatering and pressure relief wells were installed94 within the excavation adjacent to the two support walls. Figure 4.2.8 shows the locations of these pressure relief wells. Unfortunately, records of pumping rates and well operation are not available [Whelan, 1995]. However, the effects of the dewatering/pressure relief activities were measured by the four piezometers (VWPZ-108, VWPZ-

---

94 Each well consists of a 6-inch diameter PVC pipe extending 1.5-m [5.0-ft] into the bedrock. The pipe was screened above and below the Boston Blue Clay layer.
107, VWPZ-053, and VWPZ-106, Figure 4.2.2) which extend into the lower aquifer at ISS-4. The data collected at these four piezometers are reported in Figure 4.2.9.

The observed fluctuations in the pore pressure corresponded with pumping activities documented in engineers' reports and caused significant reduction and fluctuations in the pore pressure within the lower aquifer. Whelan [1995] has verified that these fluctuations are caused by pumping and are not influenced by other construction activities. The high hydraulic diffusivities in the underlying glacial deposits caused the pore pressure fluctuations to be transmitted rapidly throughout the area resulting in virtually identical readings from the four piezometers covered a distance of over 120-m wide at ISS-4. The largest drop in the piezometric head of 10.7-m within the lower aquifer was reported in January of 1993 prior to any excavation activities. This event corresponded with the initiation of pumping activities at ISS-4 documented in the engineer's report. During the remaining excavation stages, the drop in piezometric head fluctuates between 3.0-m to 7.6-m below the initial level.

The reduction of pore pressure within the lower aquifer due to the pumping activities can cause consolidation within the Boston Blue Clay and hence result in additional non-extraction induced settlements. The Base Case, presented in Section 4.2.6, incorporates this pore pressure relief in the lower aquifer. The next section, Section 4.2.5, describes the basic numerical model used for finite element analyses.

4.2.5 Finite Element Model

The numerical models used for the Base and Revised analyses are based on the site description, soil profile, and construction activities described in the
previous sections. The general description of the numerical model in terms of the finite element mesh, element type, element properties, and simulation of construction activities for the Base Case are described in this section; description for the Revised Case are presented in Section 4.8.

The numerical analyses assume that Section ISS-4 can be simulated as a plane strain problem. The central portion of the finite element mesh used for the Base case analysis is shown as Figure 4.2.10. The mesh covers the entire excavation width and extends 400.0-m laterally from the two support walls in order to eliminate the lateral boundary effects. The base of the Base Case mesh corresponds to the interface between the Boston Blue Clay stratum and the glacial deposits\footnote{The mesh for the Revised Case also include the underlying till and bedrock with the base located 10-m below this Base Case.}. Both lateral (x-direction) and vertical (y-direction) displacement constraints are imposed at the base of the mesh while the lateral limits of the mesh (400-m from the wall) are only constrained in the x-direction displacements. The seven excavation tiers as well as the six tiers of tiebacks described by Whelan [1995] are also incorporated in this finite element mesh.

Since records of pumping rates at the relief wells were not available, the measured pore pressures in the lower aquifer, which directly reflect the pumping activities, are used to define the pore pressure boundary at the base of the mesh throughout the construction period.

A total of 3,229 finite elements (of three types: mixed, solid, and spring) are included in this finite element mesh (Figure 4.2.10). The simulation of coupled fluid flow and deformation in the saturated soil is achieved using mixed isoparametric elements with 8-displacement nodes and 4-corner pore pressure nodes. The dry fill, north diaphragm wall, and the south sheet pile
wall are all modeled by solid 8-noded isoparametric elements. The tiebacks are modeled by a series of one-dimensional spring elements.

The four main soil strata (miscellaneous fill, cohesive fill, organic deposits, and the Boston Blue clay) are modeled by mixed elements, with effective stress soil models and properties. Table 4.2.5 summarizes the material properties and soil models used for each of these layers. The miscellaneous fill is modeled using a simple elasto-plastic model with a Drucker-Prager failure criterion and a non-associated flow rule (with zero dilation, referred to as an EP-DP model).

The cohesive fill and the organic deposit have a much lower permeability than the miscellaneous fill. Since these two strata are likely to experience partial drainage during the excavation period, the two strata are modeled as an Elastic-Perfectly Plastic material with undrained shear strength and stiffness proportional to the in-situ vertical effective stress (EP-von Mises). The input parameters for these two strata are also included in Table 4.2.5.

The behavior of the Boston Blue clay is represented by the MIT-E3 soil model described in Chapters 2 and 3. The input material constants for Boston Blue Clay are identical to those used in the parametric analyses in Chapter 3 and were originally based on laboratory tests on resedimented BBC (see Table 3.1.1). The stress history profile for this site, shown Figure 4.2.5a, is used to define the initial-clay OCR profile. Table 4.2.6 summarizes the actual OCR profile selected for the finite element analysis\textsuperscript{96}. The Base Case analysis assumes a constant average permeability of $k = 4.3 \times 10^{-5}$ m/day for the Boston Blue Clay stratum. This selected property, based on extensive laboratory tests

\textsuperscript{96} The numerical analyses assume constant OCR within any given finite element; therefore, the stress history profile used in the numerical analysis is step function (Table 4.2.6) rather than a continuous smooth function (Figure 4.2.5).
performed at the MIT test site in Saugus [Chantous, 1982], is typical for the lower BBC, but tends to underestimate the partial drainage within the overconsolidated crust.

The two support walls are modeled as elastic materials with properties listed in Table 4.2.7. These properties correspond to the theoretical bending and axial stiffnesses of the walls. The finite element model assumes that the walls are wished-in-place therefore cause no disturbance to the surrounding soil. Field measurements of pore pressure within the upper and lower aquifers and surface settlements did not reflect signs of disturbance due to the installation of the sheetpile wall and the diaphragm wall. The model assumes that there is no slippage at the soil-wall interface, and hence, the shear resistance is controlled by the strength of the soil adjacent to the wall. This assumption is reasonable for the cast-in-place diaphragm wall, but may be non-conservative for the driven sheetpile wall due to large pore pressures induced by the driving process.

Elastic one-dimensional spring elements are used to model the elongation of the tieback tendons (see Section 2.4). The finite element model assumes that the fixed lengths of the tiebacks remain bonded with the surrounding soil. This assumption is reasonable since the results of the proof tests on all the tiebacks assure minimal slippage in the bonded zone. Elastic properties used in the numerical model for the tiebacks are also included in Table 4.2.7.

The first seven (of eight) construction stages described by Whelan are incorporated into the numerical model with minor modifications. The last step (Step 8) is omitted in the numerical analyses because it consists of

---

97 Pore pressure increases within the clay are likely due to the driving of sheetpile walls [Finno and Harahap, 1991]; however, there were no pore pressure measurements within the BBC stratum at the south wall.
pouring invert over the subgrade and does not involve any removal of soil nor installation of tiebacks. Table 4.2.8 compares the seven construction steps included in the Base and Revised analyses with the sequence described by Whelan.

For each construction step, the numerical model divides the step into two parts: (a) The first part consists of undrained (instantaneous) excavation and/or lock-off of tiebacks; and (b) the second part accounts for the partial drainage (construction duration). Besides dividing each step into two parts, the numerical model also contains slight changes in the construction activities within Steps 2 and 3 defined by Whelan. Figures 4.2.11(a) to 4.2.11(c) illustrate the construction activities incorporated in the numerical model. Overall, the numerical model includes all the activities in the same sequence as outlined by Whelan (see Figure 4.2.7).

4.2.6 Base Case Analysis

The Base Case analysis assumes the pore pressure at the top of the glacial deposit (i.e. base of the clay stratum) is influenced by the pumping activities at the site. Figure 4.2.12 confirms that the assumed pore pressure fluctuations (based on the measured piezometric heads at the four deep piezometers) are in reasonable agreement with the measured data. Figures 4.2.13 to 4.2.20 show the wall deflections and surface displacements during each of the seven construction steps and the corresponding field measurements. In addition to the deformation patterns at every construction stage, fluctuations in pore pressures, heave within excavations, and vertical and lateral deformations during the entire construction period are included in Figures 4.2.21 to 4.2.36. Table 4.2.9 indicates the type of instrumentation data and information reported in each of these figures.
4.2.6.1 Wall Deflections and Surface Settlements at Each Construction Step

The Base Case analysis assumes that the first stage of excavation can be subdivided into twq substeps: 1a) excavation of South wall trench on March 22, 1993 [CD 0] (Figure 4.2.13) with partial drainage until May 7, and 1b) excavation of North wall trench on May 7, 1993 [CD 46] (Figure 4.2.14) with partial drainage occurring until June 5, 1993 [CD 75]. The results show the following:

1. The analysis predicts inward cantilever movements of both the North and South walls due to the excavation of trenches in steps 1S and 1N. The South wall cantilevers approximately 4.5-cm during this period [CD 75], while the maximum North wall deflection is 2-cm [CD 75] (Figure 4.2.14).

2. The predicted cantilever deflections are larger than those measured at the South Wall (INC-101) where $\delta_{W(top)} = 3$-cm was measured, although there is very good agreement with lateral deflections measured by IPE-113 located 8-m behind the wall. The data measured in the North wall (INC-102) are subject to a zeroing error shown in Figure 4.2.13, where the initial reading show a 1-cm vertical rigid body translation of the North Wall. Accounting for this zero error, the maximum cantilever movements measured at Stage 1Nb [CD 95] (Figure 4.2.14), is approximately 1.5-cm, which is slightly smaller than the predicted $\delta_{W(max)} = 2$-cm.

3. The Base Case analysis predicts some settlements of the retained soil during the first stage of excavation. Far field settlement at 45-m from the South wall is $\delta_V = 1.3$-cm, and $\delta_V = 0.9$-cm [CD 75, Figure 4.2.14] at a similar distance from the North wall. These movements at a distance from the excavation are related to changes in the water pressures

---

98 Whelan [1995] noted that the measured values are likely to underestimate the actual movements, as both INC-102 and INC-101 were initialized tow days after excavation commenced.
within the underlying till as the result of pumping activities\textsuperscript{99}. During Steps 1S and 1N, the analysis also predicts 3.9-cm of settlement of the South wall, while no vertical movements occurs at the North wall as the Base Case assumes that the underlying till and rock layers are rigid.

4. There are no direct measurements of vertical wall movements for either the North or South walls. However, the surface settlements measured at 3 points beyond both walls are generally in good agreement with the numerical predictions.

Stage 2 of the analysis comprises the application of lock-off load at the first tier of tiebacks along the North and South walls and the excavation of the North and Central section (tier 2). These events are assumed to have occurred on June 5, 1993 [CD 75] and allowed 26 days of partial drainage (July 1, 1992, CD 101]. The resulting surface settlements and wall deflections are presented in Figure 4.2.15, and it shows the following:

1. The analysis showed that on June 5, the application of 400-kN/m at the North wall is sufficient to pull the top of the wall into the retained soil beyond its original location, while there is a maximum inward deflection \( \delta_{w(max)} = 2.1\text{-cm} \) at a depth of 7.6-m. The analysis predicts small wall deflections away from the excavation due to partial drainage (results of CD 75 vs. CD 101, Figure 4.2.15). The measured data showed a very similar mode shape of the diaphragm wall, with maximum inward deflections of 1.3-cm at depth 10 to 12-m, and total lateral movement of 1-cm away from the excavation at the top of the wall.

2. There are large differences between the predicted and measured settlements behind the North wall. The analysis show a small increase in the far field settlement (\( \delta_y = 1\text{-cm} \) at \( x = 45\text{-m} \) and settlements less than 1.4-cm close to the excavation with no vertical movements of the North wall. In contrast, the measured settlements increased dramatically to 4.5 to 4-cm at locations 7-m and 12-m from the wall.

\textsuperscript{99} An additional analysis was performed which does not include the pumping activities in the underlying till. The results show negligible surface settlements at some distance from the wall.
respectively. However, far field settlement (at x = 45-m) measured on June 25 is less than 0.7-cm.

3. The results in Figure 4.2.15 show that the South wall (SPW) is also pulled back from the excavation by the prestress of the level 1 (1-S) tiebacks. After locking off a load close to 200 kN/m on June 5, the wall has a maximum inward deflection of 2.1-cm at a depth of 7.6-m while the top of the wall returns to its original alignment after 26 days of partial drainage (July 1). The predicted movement of the South wall are in reasonable agreement with INC-101 data measured on June 9; however, the measurement on June 29 showed a large outward wall movement where maximum wall deflection increased to close to 4-cm. This large increase between June 9 and June 29 was not described by the analysis. This can be accounted, in part, by an offset in the finite element model and actual construction schedule where further excavation took place during this period but is included within the next numerical excavation step (cf. Table 4.2.8). The analysis also predicts pullback of the upper part of IPE-113 located behind the South wall due to the preload of the tieback. However, this effect is not observed in the measured data as IPE-113 show slight disturbance by the installation of tieback.

4. Predictions of surface settlements behind the South wall are generally in good agreement with measured data at locations 12-m, 17-m, and 26-m from the wall. Both the predictions and measured data showed small increases in settlement from Step 1 with predictions underestimating the actual settlements by 0.5 to 1-cm.

Stage 3 of the base case analysis comprises excavation of soil down to the organic layer on the southern half of the section (Figure 4.2.16) together with lock-off for the second tier of tiebacks on the North wall (2-N) on July 1 (CD 101) followed by 39 days of partial drainage (Aug. 9, 1993, CD 140). The predictions of wall deflections for the North wall continue to be in excellent agreement with the measured data. Preload of the 2-N tiebacks caused further pullback of the wall with lateral movements of 6.2-cm away from the
excavation at the top [CD 140]. The numerical predictions for surface settlements are consistent with the wall deflections with movements exceeding the original ground elevation 5-m from the wall, while the largest settlement occur in the far field ($\delta_v = 1$-cm at 45-m). In contrast, the measurements show large settlements occurring up to 30-m from the wall, with maximum recorded values of 9-cm at 7-m from the wall. The data follow the same trends observed in Step 2 and appear to be largely unrelated to excavation and preload activities simulated in the analysis. In the far field, the predicted settlements are in excellent agreement with the measured data. Some possible causes include tieback installation and soil properties which will be addressed later.

The step 3 predictions for the South wall reflect only the excavation event itself (i.e. there is no simulation of tieback anchor installation) (Figure 4.2.16). The analysis predicts a small inward movement of the wall with maximum movement of 3.7-cm at a depth of 6.8-m on Aug. 9 (CD 140). The predictions are in reasonable agreement with the measured data on July 9. However, the field measurements show continued inward movement of the wall up to a maximum deflection of 9-cm on July 21. Similar trend was also observed in the surface settlements behind the South wall. The predictions are in good agreement with settlement data measured on July 8 but greatly underestimate settlements observed on Aug. 3, 1993 by close to 2-cm. However, the predicted settlement at 26-m from the wall is in excellent agreement with the measured data.

Step 4 (Figure 4.2.17) comprises the excavation of tier 4, near the North wall, and lock-off of level 2 tiebacks at the South wall (2-S). This stage generates small lateral wall deflections (up to about 0.3-cm of additional inward movement toward the excavation at the top of North wall) and
surface settlements behind the North (diaphragm) wall. Hence, there is no change in the previous comparison between predicted performance and measured behavior. However, predictions for the South wall are primarily related to the lock-off of the 2-S tiebacks. The analysis predicts large pullback deflections during this process, such that the maximum inward wall deflection reduces to 2.2-cm (at 10.8-m) while the top of the wall is pulled back into the retained soil beyond its original alignment \( \delta_{w_{\text{top}}} = 1.2\)-cm. The field measurements show that lock-off of the 2-S tieback also pulls the wall back into the soil (from \( \delta_{w_{\text{max}}} = 9\)-cm before lock-off the tieback (July 21, Figure 4.2.16) to \( \delta_{w_{\text{max}}} = 7\)-cm after the lock-off (Aug. 9)). However, the measured data show very large continued inward movements of the wall from 7-cm to 14-cm on September 2, 1993. Although IPE-113, located 8-m from INC-101, also reported continued movement toward the excavation, the amount of increase, less than 1-cm, was significantly less than the observed movement at INC-101 within the same period. The numerical prediction of surface settlement behind the South wall in Step 4 showed close to 2-cm of upward movement at \( x = 5\)-m, which is consistent with predictions of wall deflections; but measurements close to the excavation show continued increase in the surface settlements which are about 3.5-cm more than the predicted settlement. Interestingly, at 26-m from the wall, the predicted settlement continue to be in excellent agreement with measured settlement.

Incremental wall deflections and surface settlements in Step 5 (Figure 4.2.18) are generally small compared to the preceding steps and exhibit similar differences compared to measure data as discussed previously compared; therefore, the results will not be discussed in detail at this stage. The analyses of Steps 6 and 7 (Figures 4.2.19 and 4.2.20) do not account for construction activities associated with installation of tiedown anchors (jet grouting) for the
base slab of the boat structure; therefore, the predictions and the measured values are not quite comparable. Nevertheless, the large differences between the predictions and measurements of the surface settlements close to the excavation (behind both South and North walls) and South wall deflections continue to increase as the excavation progress, while the predicted North wall deflections and settlements at a distance from both walls continue to be in excellent agreement with the measured data.

4.2.6.2 Time History Comparisons

Figures 4.2.21 to 4.2.36 show time history comparisons of pore pressure, vertical movements, and lateral movements at location of the field instruments (Figure 4.2.2). The predicted and measured piezometric elevations at various locations around the excavation as functions of time are compared in Figures 4.2.21 to 4.2.26. The results from 4 piezometers installed in the underlying till/rock were used to define boundary conditions used in the FE analysis using the simplified pore pressure histories shown in Figure 4.2.12. The piezometric elevation within the overlying fill remains rather constant as measured by observation wells OW-016 and OW-002 on the North side of the excavation (Figure 4.2.21) and at OSPZ-106 at the South side of the excavation (Figure 4.2.23). The vibrating wire piezometer, VWPZ 67/68, located at the top of the clay stratum (depth 12.4-m) at 12-m north of the North wall (Figure 4.2.22) showed relative small changes in pore pressure prior to mid-September, which are consistent with the numerical predictions. However, a steady decrease in piezometric elevation was reported at this location after September, 1993, which was not predicted by the analysis. Though this change coincided with the installation of tiedowns, this behavior is not likely to be related to the installation of tiedown anchors within the
excavation, rather, the observed decrease in pore pressure at this location may be related to its proximity to 1-N tiebacks (Figure 4.2.3) which could be a potential drainage path.

Figures 4.2.24 to 4.2.26 compare the predicted and measured piezometric elevations at the three pairs of piezometers located inside the excavation below the excavation grade at approximate depths of 18-m and 24-m (VWPZ 135/136, VWPZ 133/134, and VWPZ 131/132). In general, these data show the following:

1. All three pairs of piezometers show large decreases in piezometric elevation during excavation. Minimum values prior to the installation of tiedowns are typically in the range of 20 to 23-m (El. 65 to 75-ft) which represents a net decrease of 9.3 to 6.3-m with a typical excavation depth of H = 9-m across the section.

2. During the installation of the tiedowns, there are few cycles of gradual increase / recovery of piezometric elevation with final values ranging from H = 22 to 25-m. Very high pore pressures measured at VWPZ 135/136 after mid-September are likely to be associated with its proximity to jet grouting used to secure the tiedown anchors.

3. The analysis overestimates the change in piezometric elevation at each excavation step. At the same reference time, July 1, 1993 (CD 101), the predicted piezometric elevation range from 18 to 20-m while the measured elevation range from 22 to 24-m. This result can be linked to the consolidation (stiffness and flow) of the soils below the excavated grade. The numerical analysis treats each excavation tier as an undrained process, which generates significant (negative) excess pore pressure below the excavated grade (seen by the drops in pore pressure shown in Figures 4.2.24 to 4.2.26). Thereafter, small amounts of partial drainage cause a small increase in pore pressure (piezometric elevation). Underestimation of the measured piezometric elevation occurs because the analysis underestimates the rate of consolidation in the cohesive fill, organic silt and upper crust of BBC. The results in
these figures suggest that revisions in the permeability and/or stiffness properties of these layers may be necessary.

Figures 4.2.27 to 4.2.30 compare the predicted and measured vertical displacements (positive displacement indicate heave) below the excavated grade at four locations of the multi-point heave gauges (MPHG) located within the excavation (MPHG 110, 109, 501, and 107; Figure 4.2.2). At each instrument location, vertical displacements at three depths corresponding to the base, middle, and top of the BBC deposits are included. The measured data show three distinct phases of behavior:

(i) small settlements of the clay, typically of the order of 1-cm\(^{100}\) at the top of the layer, which develop prior to excavation (from Jan. to June, 1993). These movements are most probably related to pumping from the underlying till and rock.

(ii) Between July and mid-September, the stress reduction caused by the excavation generates swelling within the clay layer and vertical heave displacements at all three depths. Most of the records show a plateau of maximum heave displacement of \(\delta_{hv(max)} = 3.5\) to 4.5-cm (at the top of the clay).

(iii) The installation of tiedown anchors (and associated jet grouting) promotes further heave movements after mid-September.

The numerical analysis predicts negligible settlements due to pumping and underestimates the magnitude and rate of heave within the clay layer caused by excavation. The predicted average rate of heave between July and September is approximately 0.008 cm/day at the top of the clay, corresponding to about 1/3 of the measured heave rate. This result confirms previous observations that the base case analysis underestimates the field consolidation rates, especially within the upper half of the clay.

\(^{100}\) Data from MPHG-107 show up to 2-cm of settlement.
Figures 4.2.31 to 3.2.33 show the time histories of lateral movements at selected depths on INC-102, INC-101, and IPE-113. As described in the previous section, the numerical model captures the lateral movements at the North diaphragm wall (Figure 4.2.31); however, the numerical under-predictions the lateral movements at the South wall (Figures 4.2.32 and 4.2.33).

The time histories of surface settlements of the retained soil and vertical movements within the clay layer measured by probe extensometer (IPE-113) are shown in Figures 4.2.34 to 4.2.36. The results for the north side of the excavation (Figure 4.2.34) show good agreement between predicted and measured far field movements ($x > 40$-m), while the analysis deviates completely from the measured behavior after the installation of 1-N tiebacks. This results suggests that the tieback installation processes have a major effect on the measured ground settlement. Whelan (1995, p. 56) noted that "during drilling (of tiebacks behind the North wall), air and wash water were seen escaping from adjacent tieback holes and from tieback holes in the higher tiers, indicating communication of water and pressurized air through the Boston Blue Clay". Considering that the Boston Blue Clay deposit is highly impermeable, this observation implies that "the clay was being fractured under pressure (during tieback installation), and hence greatly disturbed by the use of rock bit and pressurized air". The large surface settlement may then be related to the ground loss caused by the tieback installation process. In fact, the settlement histories at the three settlement points closest to the North wall (DMP2-006, DMP2-107, and DMP4-120, at 7, 10, and 12-m from the North wall, respectively) are very similar in magnitude. This similarity reinforces the suspicion that the additional settlements are likely to be due to tieback installation and not a result of excavation since all three levels of
tiebacks extend beneath these three settlement points (see Figure 4.2.3). Only the top two levels of tiebacks extend below DMP2-105 (at 28-m from the North wall), and hence, it experiences a smaller settlement. Settlement points DMP2-104 and DMP2-070 (at x = 42-m and 45-m) are located beyond the influence of the tiebacks; therefore, show no correlation with tieback installation. The predicted settlements at DMP2-104 and DMP2-070 also show excellent agreement with measured data.

Assuming that ground loss due to tieback installation is equal to the volume of the drilled tieback hole\(^{101}\), the soil within 28-m from the wall is expected to settle 3.4-cm, while settlements of 2.1-cm and 0.8-cm are expected for 28-m < x ≤ 32-m and 32-m < x ≤ 36-m, respectively. This tieback installation-induced settlement is well below the observed settlements at the end of excavation (\(\delta_{v,\text{max}}\) ≈ 14-cm); however, this magnitude correlates well with the observed settlement at the time of tieback installation. For example, after the installation of the top two tiers of tiebacks in early June, settlements close to 2-cm were observed at DMP4-120 and DMP2-006; coincidentally, the tieback installation-induced settlement estimated after installation of 2 tiers is also 2.1-cm. The continued increase in settlement well after tieback installation and no significance excavation activities suggest that the fracture of the clay during tieback installation may have created additional drainage paths thus altering the consolidation behavior of the BBC.

Similar comparisons for surface settlements on the South side (Figure 4.2.35) show much better agreement between predicted and measured data through early July. During the period of Mar. 22 to July 1 (CD 0 to CD 101), most of the difference between predictions and measurements can be

\(^{101}\) Assuming that the hole has a diameter of 15.24-cm which is equal to the casing used during installation of tiebacks. Since seepage of water and air was observed during tieback installation, this assumption is likely to be non-conservative.
accounted by the initial settlements due to groundwater pumping. The installation of 2-S tiebacks around CD 100 appears to correlate quite closely with increased rates of measured settlements close to the excavation. Based on tieback installation logs, the tiebacks in the vicinity of INC-101 were subjected to at least 1 retest which suggests that the tiebacks failed the initial proof tests and was regrounded and retested. Therefore, the observed settlement behavior may be influenced by the tieback installation procedure. The large difference between predicted and measured behavior after July 1 may also be related to other factors relating to soil properties (notably underestimation of consolidation rates in the clay layer, or overestimation of stiffness properties in the organic silt and cohesive fill).

Figure 4.2.36 shows the vertical movements at the top, middle, and base of the clay layer measured by the probe extensometer IPE-113. The data from the middle and base of the clay layer show small settlements occurring prior to July; thereafter, the data appear to be affected by the installation of the 2-S and 3-S tiebacks. In general, both points show a net vertical settlement of less than 1-cm. In contrast, the vertical settlement at the top of the clay (at Magnet 10) increases to 4-cm by August prior to the installation of 3-S tiebacks. The numerical predictions show no movements in the lower half of the clay with approximately 1-cm of settlement at the top of the clay (underestimates the actual settlement observed after August by at least 3-cm).

4.2.7 Assessment of Base Case Analysis

The preceding comparisons between the Base Case Analysis and measurements from section ISS-4 in South Boston can be summarized as follows:
1. Lateral deflections of the North (diaphragm) wall are well predicted at all stages of the excavations, with support provided by three levels of prestressed tieback anchors in the underlying rock. Tieback preload forces pulled the wall back up to 12-cm behind its original alignment.

2. Surface settlements on the North side of the excavation are grossly underpredicted within 30-m of the North wall while far field movements are well described at distances greater than 40 to 50-m. The magnitude of measured surface settlements close to the wall ($\delta_v = 14$-cm at $H = 12.5$-m, $x = 7$-m, $\delta_v/H = 1.1\%$) are very large for a diaphragm wall supported excavation. There appears to be a link between these settlements and the installation of the tieback anchors.

3. The Base Case analysis predicts reasonable magnitudes of the initial cantilever movements of the South (sheetpile) wall but greatly underestimates the measured inward lateral deflections that occur after tieback lock-off (3 levels of tieback anchors grouted in the clay crust). The analyses consistently predict pullback of the wall during tieback preload and lock-off, while the measurements show small pullback movements that are subsequently overwhelmed by continued inward deflection of the wall. The measured wall deflections reach a maximum value of $\delta_{w(max)} = 14$-cm close to the final excavation grade.

4. Surface settlements on the South side of the excavation are underpredicted by the analysis. In part, this reflects the discrepancies in predicted and measured wall deflections. However, there is supporting settlement data within the clay which shows that the Base Case analysis underestimates the compression of the upper half of the clay deposit.

5. The vertical displacements and pore pressures of the clay layer below the base of the excavation was measured by a series of probe extensometers and vibrating wire piezometers. These data show clearly the effects of groundwater pumping from the underlying till and monitor the dependent swelling of the clay due to removal of overburden stress. The Base Case analysis consistently underestimates the consolidation rates within the clay layer, generating small amounts of heave and lower pore pressures, compared with the measured data.
Overall, the Base Case analysis has generated a large number of questions such as:

a. the capabilities of finite element analysis to simulate complex construction process such as bracing using tieback anchors.

b. the importance of construction details on controlling ground movements (e.g. detailed documentation of groundwater pumping and installation of tieback anchor).

c. the reliability of the representation of soil properties used in the numerical analysis.

Although the Base Case analysis achieved good predictions of lateral deflections for the North wall, the inconsistencies and discrepancies with all other aspects of the measured performance requires further investigation.

This section reviews the assumptions used in the Base Case finite element model and hence, proposes a series of modifications to be incorporated in a Revised analysis (Section 4.2.8). There are two main groups of assumptions that need to be considered 1) general finite element model approximations such as boundary conditions and initial state, and 2) input parameters for constitutive behavior (mechanical and hydraulic properties).

### 4.2.7.1 Approximations in Finite Element Model

There are numerous approximations used in the finite element model ranging from the selection of boundary conditions, to the representation of the construction schedule (as a series of discrete steps). The analyses presented in this thesis assume plane strain conditions and focus on well instrumented cross-sections where the excavation history is carefully defined. In the previous study of South Cove, Section 4.1, underprediction of sheetpile
wall deformations were attributed, in part, to longitudinal deformations (i.e. out of plane movements of the wall) which were measured during construction. There are no such data for this project, although the Base Case analysis shows a similar underestimation of the South sheet pile wall deflections\textsuperscript{102}.

The base case analysis assumes that the underlying glacial deposits and rock strata do not deform during the excavation and can be treated as a rigid base. However, these materials have a much higher permeability than the overlying clay, and are treated as a pore pressure boundary which varies with time according to pumping from deep wells shown in Figure 4.2.8. One consequence of these assumptions is that there are effectively no vertical displacements\textsuperscript{103} of the North diaphragm wall which extends to the rigid base. Although these assumptions are certainly reasonable, pre-stressing of the steeply inclined (45°) rock tiebacks includes large vertical force component would provide an additional mechanism for generating surface settlements in the retained soil. Such movements would become significant if the wall were embedded in the till or upper weather rock, rather than keyed into sound rock. Detailed records presented by Whelan (1995, Figure C4) show the embedment depths of individual diaphragm wall panels which vary by up to 2.5-m over the 46-m length of the wall. In each case, the slurry trench was excavated up to 0.6-m into the underlying rock. Previous studies (Whittle et al., 1993) have found rather small contrasts between the stiffness and strength properties of glacial till and weathered argillite. Hence, a Revised analysis incorporating these layers in the finite element model can assess whether

\textsuperscript{102} Indeed, it is difficult to characterize the South Boston Project as a plane strain problem, as the width of the excavation is of comparable magnitude to characteristic length dimensions over which there are significant changes in geometry (cf. plan of project, Figure 4.2.1)

\textsuperscript{103} Apart from elastic compression of the wall itself.
vertical deformations of the wall can explain some of the measured surface settlements on the northern side of the excavation\textsuperscript{104}.

The Base Case analysis also makes a series of assumptions regarding the interactions of the structural elements and surrounding soil, which are difficult to validate but have an important influence on the performance of the excavation:

1. wall and tieback anchors are wished-in-place, and cause no change in stresses or properties of the surrounding soil.
2. there is no slippage between the fixed anchor and the surrounding soil during loading. The fixed anchor is represented by a series of elastic springs connected at multiple points to the soil.
3. the interface shear resistance between the walls (diaphragm and sheetpile) and the adjacent soil is controlled by the shear resistance of the soil itself (i.e., failure takes place within the soil, not by sliding at the interface).

The installation of cast in-situ diaphragm walls can induce small settlements of the surrounding soil; however, negligible settlement was observed during the installation of the North diaphragm wall. The interface between a cast-in-situ wall and soil is usually quite rough and therefore, the current model assumption regarding interface shear resistance is reasonable.

Sheetpile walls are driven into position, causing displacements of the surrounding soil and inducing large excess pore pressures within low permeability soft clays [e.g. Finno and Harahap, 1991]. This installation process can indeed cause a significant disturbance of the surrounding soil

\textsuperscript{104} Inclusion of the underlying glacial till and bedrock is unlikely to yield the level of surface settlements observed in the field since the sum of the vertical force component of the 3 levels of tiebacks is well below the force required to cause 14-cm of settlement in the wall as well as settlements in the soil within 40-m from the wall.
modifying both the stress field and soil properties adjacent to the wall. The
disturbance caused by sheetpile wall installation is closely analogous to the
problems caused by driving foundation piles. Extensive research on this topic
at MIT has led to the development of strain path analyses [Baligh, 1985] for
simulating installation disturbance in clays. These analyses show how
undrained shearing during pile installation alters effective stress in the
surrounding soil and generates excess pore pressures that can extend up to 50
to 100 pile radii. After installation, the dissipation of excess pore pressure is
accompanied by changes in effective stresses, a process referred to as pile set-
up [Azzouz et al., 1990, Whittle, 1992]. After full dissipation of the
installation pore pressures, the analyses show a significant net decrease in the
shear resistance of soil adjacent to the pile shaft in soft (low OCR) clays.
Similar effects can be expected for sheetpile walls, although no comparable
analyses have yet been performed\textsuperscript{105}. Conventional geotechnical practice
frequently assumes a reduced interface friction angle between the steel
sheetpile and the soil (typically $\gamma' = 0.5$ to 0.33 $\phi'$). However, there is no
theoretical basis for this assumption, although superficially, it has a similar
net effect on shaft resistance to the strain path models described above.

Overall, the assumptions of no installation disturbance and full soil
shear strength along the sheetpile-soil interface are not realistic and are non-
conservative as the finite element model will tend to underestimate wall
deflections and ground movements. Incorporation of the full effect of
sheetpile installation with the strain path-type analysis is beyond the scope of
this research. However, results from the Base Case suggests that this area is
worth further investigation in order to obtained better predictions of the

\textsuperscript{105} Recent work by Sagaseta et al., 1995, includes predictions of ground deformations due to
sheetpile installation. These have been validated by comparison with field data reported by
behavior of sheetpile walls and corresponding settlements close to the excavation. However, the South Boston ISS4 section may not be a good candidate for this exercise since there are no pore pressure monitoring points within the underlying soft clay outside of the excavation.

The installation of the tieback anchors in the clay is a complex process involving several stages of pressure grouting and anchor proof testing. In general, the pressure grouting replicates a cavity expansion type of deformation within the surrounding soil. This process causes local densification and/or net increases in the stress field\textsuperscript{106}. Hence, it is likely to increase the shear strength of the surrounding soil. Furthermore, proof testing of the anchor itself demonstrates that there is minimal slippage along the fixed anchor length; therefore, the modeling assumptions for the fixed tieback anchor appear to be conservative (i.e. will tend to overestimate the ground movements).

Drilling of tieback holes for the tendons is not a process considered in the analysis at all. However, there is clear evidence from the South Boston case study [Whelan, 1995] that the procedure used to drill the North wall tiebacks were highly disruptive and may account for a significant amount of measured surface settlement on the northern side of the excavation. As mentioned in the previous section, assuming full ground loss (i.e. equivalent surface settlement assuming loss of soil equal to the volume of the tieback hole), settlement of 3.4-cm is expected within 28-m from the North wall (with all 3 tiebacks directly beneath the surface). This effect can only account for a small portion of the 14-cm settlement observed behind the North wall. In addition to ground loss, the installation of tiebacks may have created additional drainage paths within the clay stratum by fracturing the clay;

\textsuperscript{106} The installation of tieback anchors may also induce fractures within the clay.
hence, creating additional drainage paths which would accelerate consolidation and attribute the large and steady increasing settlements observed behind the North wall.

4.2.7.2 Material Properties

Apart from the main deposit of BBC, there are three distinct overlying layers (miscellaneous fill, cohesive fill, and organic silt) included in the Base Case analysis\textsuperscript{107}. These upper layers are modeled using relatively simple soil models (elastic-perfectly plastic) with input parameters selected from very limited experimental data. The miscellaneous fill is free draining, while fluid flow through the cohesive fill and organic silt is specified by a constant coefficient of permeability (Table 4.2.5). The predictions of the initial cantilever movements (Figures 4.2.13 and 4.2.14) are generally in reasonable agreement with measured wall deflections and hence, there is no cause for speculating on re-selecting the stiffness parameters of these layers.

The mechanical properties of Boston Blue Clay are represented by the MIT-E3 model, with input parameters selected by Whittle (1987), and the initial stress history defined from pre-consolidation pressures measured on samples from section ISS-4 [Whelan, 1995, see Figure 4.2.5]. The Base Case analysis also assumes a constant hydraulic conductivity of the clay \(k = 4.3 \times 10^{-5} \text{ m/day}\). The following paragraphs discuss some limitations of these assumptions.

The MIT-E3 model is based on the concept of normalized soil behavior, where the stiffness and strength properties are proportional to the current effective stress state at a given OCR. The input material constants for BBC

\textsuperscript{107} Properties of the underlying glacial deposits and the weathered argillite are discussed in Section 4.2.8 for the Revised Analysis.
were derived originally from laboratory tests on resedimented BBC (Whittle, 1987). Recent research at MIT has i) refined the measurement of resedimented BBC properties [Seah, 1990; Sheehan, 1991], ii) identified important differences between the normalized strength and stiffness properties of natural and resedimented BBC [Estabrook, 1991], and iii) confirmed the wide variations in stress history profiles of BBC within the Boston basin. These studies all suggest limitations in the current set of input parameters used by the MIT-E3 model. Two specific examples can be cited:

1. The MIT-E3 model overpredicts the undrained strength ratio \( \frac{\sigma_u}{\sigma'_v} \) of natural BBC. This effect is shown in the MBTA Transitway project presented in Section 4.3. This over-prediction of strength, especially for slightly overconsolidated soils, could yield significant reduction in displacement (see Section 3.3).

2. Compression properties of natural BBC vary significantly with depth according to the deposit conditions. Ladd et al. [1994] demonstrated how refined compression properties of BBC based on lab test data could be incorporated in this type of finite element analysis. However, the Base Case predictions of far field surface settlement (at \( x > 40\text{-m} \)) at ISS4 are generally in good agreement with the measured data. This result suggests that the refinement of compression properties may not be warranted for this study.

Overestimation of undrained shear strength parameters is a major problem in assessing excavation performance, especially for situations where there is a low factor of safety against basal stability. This result is seen clearly in the Group D numerical experiments presented in Section 3.3, where there are large differences in predicted displacements for relatively small variations in OCR in the range of OCR = 1.0 to 1.5. Uncertainties in the undrained shear
strength within the clay crust, however, have a much smaller effect on predicted ground movements (Group F analyses, Figure 3.3.38). The overestimation of undrained strength properties for the lower soft clay at ISS4 may have significant impact as the measured deep-seated lateral movements (below 20-m) deviates from the predictions as the excavation proceeds beyond \( H = 7.5 \)-m (see Figure 3.3.36). Therefore, future refinement of the BBC input parameters, which incorporate natural BBC behavior, for MIT-F3 is necessary in order to improve the FEA predictive capabilities.

The comparisons between Base Case predictions and measured pore pressures and vertical displacements within the clay do consistently show that the analysis underestimates the consolidation rates (especially within the clay crust). At points inside the excavation, the consolidation rates is controlled by the unloading stiffness of the soil skeleton and the hydraulic conductivity of the clay. The Base Case analysis assumes that the hydraulic conductivity is uniform throughout the clay and is isotropic. However, previous research at the MIT test site [Baligh and LeVadax, 1986; Morrison, 1984, and Ghantous, 1980] shows that the hydraulic conductivity of the clay is much higher in the crust than in the lower clay\(^{108}\) which the horizontal coefficient of permeability is significantly larger than that in the vertical direction \((k_h/k_v = 1.5 \text{ to } 2.0)\). Ladd et al. [1994], included variations of permeability in their analysis of embankment performance on BBC. However, these authors also concluded that their analysis underestimated the measured consolidation rates within the clay crust. They attributed this behavior to sand lenses within the Clay. Since comparison of piezometric elevation measured within the excavation showed differences in the

\(^{108}\) The assumed \( k = 4.3 \times 10^{-5} \text{ m/day} \) is similar to properties measured for the lower clay at Saugus.
consolidation rates, refinement of the permeability properties of the BBC should be incorporated within the Revised Case.

4.2.8 Revised Case Analysis: including underlying bedrock, revised permeability profile, and omission of Tier 2 tieback on the South Side

Figure 4.2.37 summarizes the modifications and adjustments incorporated within the Revised Analysis. In general, the Revised Case follow the same excavation schedule as the Base Case analysis (Figure 4.2.11). The three main changes are 1) inclusion of the underlying 3-m thick glacial deposits and 6.7-m\(^{109}\) of bedrock, thus extending the base of the mesh by 9.8-m, 2) refinement of the permeability properties within the BBC stratum to account for higher permeability within the clay crust, and 3) omission of the tier 2 tiebacks on the South wall (2-S) in order to examine if the integrity of these tiebacks may have attributed to the large observed deformations.

Figures 4.2.38 to 4.2.43 show the comparisons of soil movements for each construction step from Step 1N to Step 6 (CD 46 to CD 345). Comparison of Base Case and Revised Case results is shown as Figure 4.2.44. Time histories of pore pressure, heave within the excavation, horizontal displacements, and surface settlements are shown in Figures 4.2.45 to 4.2.60.

4.2.8.1 Construction Step Comparisons

Predicted wall deflections and surface settlements obtained from the Revised Case are plotted against the measured data for construction Steps 1N (May 7, 1993, CD 46) to Step 6 (Mar. 2, 1994, CD 345). At Steps 1N, 2, and 3 (Figures 4.2.38, 4.2.39, and 4.2.40), prior to the lock-off of the 2-S tiebacks, the

\(^{109}\) The North diaphragm wall is keyed 0.6-m into the underlying. The bottom of the Revised Case FE mesh extends 6.1-m below the bottom of the wall.
basic predicted wall deflections are very similar to the Base Case with a slight reduction in magnitude (by approximately 10%) (see Figure 4.2.44 for comparison between Revised Case and Base Case). The predicted surface settlements were also smaller than the Base Case. This reduction in magnitude reflects the effect of altering the permeability properties within BBC and, to a lesser extent, the inclusion of the underlying glacial deposits and bedrock. Despite of this reduction in magnitude of deformation, the predicted North wall deflections and settlements at a distance from both walls continue to be in excellent agreement with the measured data; however, the differences (between the Revised Case and measured data) in the South wall deflections and settlements close to the excavation are increased.

At Step 4 (Figure 4.2.41), the omission of the 2-S tiebacks yielded negligible changes in the predicted deformations on the South side compared to Step 3 as most of the construction activities took place near the North wall. However, even without the presence of the 2-S tiebacks, the predicted wall deflections are still significantly less than the observed displacements (close to 2.5-cm less than displacements measured on Aug. 9). Nevertheless, the predicted far-field settlements and the North wall deflections continue to remain in excellent agreement with measured data.

At Step 5 (Figure 4.2.42), the South wall moved an additional 2-cm without the presence of the 2-S tiebacks. This increase in deflection is still significantly less than the observed maximum deflection of over 14-cm recorded on September 2\textsuperscript{110}. The predicted settlements close to the excavation also underestimate the actual movements by 10-cm and 3-cm at the North and South walls, respectively. However, the North wall deflections and far-

\textsuperscript{110} Note that this large increase in wall displacement within the month of August was not observed at IPE-113, which is located only 8-m behind INC-101.
field settlements still remain in good agreement with the measured data. This pattern continues to persist through Step 6 (Figure 4.3.43).

Figure 4.2.44 shows a direct comparison of the Base Case and Revised Case analyses at construction Steps 1N, 3, 5, and 7. As shown in this figure, changes in the permeability properties within the BBC stratum yielded larger changes in the surface settlement than the wall deflections. The absence of the 2-S tiebacks alters the South wall deflection within 10-m from the excavation; but has negligible effects on deep-seated movements (more than 10-m below the excavation grade). The effect of including the underlying glacial deposits and the bedrock appear to be limited as virtually identical vertical movements were observed at the North wall.

4.2.8.2 Time History Comparisons

Time histories of piezometric elevations for the piezometers or observation wells located outside of the excavation show negligible change compared to the Base Case (Figures 4.2.45 to 4.2.47). However, the history of piezometric elevations within the excavation show significant improvement compared to the Base Case with less than 1-m of difference between the measured and predicted values at VWPZ-135/136 (Figures 4.2.48 to 4.2.50). This improvement in the predicted pore pressure reflect the effect of changes in permeability profile within the BBC; and demonstrated that this refinement is necessary for good prediction of pore pressures, especially within the excavation. Similar improvements were also observed for the heave within the excavation with increased heave predicted during July and early August (Figures 4.2.51 to 4.2.54).

As stated earlier, the changes incorporated in the Revised Case did not alter the behavior of the North diaphragm wall; hence, history of lateral
movements at INC-102 exhibit the similar trend as noted earlier (Figure 4.2.55). Though the omission of the 2-S tiebacks generated additional wall movements compared to the Base Case, the Revised Case predictions still grossly underestimates the South wall deflections (Figures 4.2.56 and 4.2.57) as well as settlements close to the excavation (Figures 4.2.58 and 4.2.59).

Although the refined permeability properties within the BBC stratum improved predictions of heave and pore pressures within the excavation, the Revised Case predictions grossly underestimates the vertical displacements measured at iPE-113 (Figure 4.2.60). The measurement indicates approximately 2-cm of settlements at the lower clay deposit, below the tiebacks, and around 6-cm of settlements at the top of the BBC strata. The history of measured vertical movements showed distinct step changes at all three depths around early-July and mid-August, which correlated with the installation of tieback. Since the lateral movements measured at IPE-113 showed significant distortion caused by the tieback installation procedure, the measured vertical movements are equally likely to reflect the same disturbance by tieback installation.

Overall, the Revised Analysis showed that the inclusion of revised permeability profile can improve heave and pore pressure predictions in the clay located within the excavation. However, the omission of the 2-S tiebacks and the inclusion of underlying glacial deposits and bedrock still could not account for the large wall movements observed at the South wall and the large settlements close to the wall.

4.9 Evaluation of the South Boston Excavation Case Study

The Base Case and Revised Case analyses demonstrated the capabilities and the limitations of the numerical models describing excavations
supported by tiebacks. The proposed analysis achieves very good predictions
of diaphragm wall deformations, but, for more flexible walls (i.e. sheetpile
wall), the current technique is not capable of attaining the same level of
performance. Possible causes include uncertainties in the strength properties
of the surrounding soil, possible deviations from the assumed plane strain
conditions, and tieback properties. Though the impacts of some of these
factors were addressed in Chapter 3 and discussed in Section 4.2.7, the exact
magnitude is rather difficult to quantify for a specific case given the limited
field data. Nevertheless, these factors generally influence the lateral
movements above the clay stratum, good and reliable predictions of the
lateral movements at and below the toe of the wall are still possible with the
current numerical technique\textsuperscript{111}.

The nature of the construction technique at this South Boston site also
dictated additional factors to be considered in the numerical analyses and the
evaluation of the numerical results. The South Cove and the South Boston
sites consist of two major differences: 1) at the South Boston site, significant
pumping in the lower aquifer took place before and during the construction
period; and 2) the South Boston site is supported by tiebacks thus involved
installation activities behind the support walls. The impact of these two
differences is most visible in the observed surface settlement troughs: In the
case of South Cove, most of the measured surface settlements can be
attributed to the excavation process; however, the settlements at the South
Boston site were dominated by tieback installation (for settlements near the
wall) and pumping in the lower aquifer (for settlements at a distance from the
wall). Surprisingly, the numerical analyses suggest that the excavation

\textsuperscript{111} Modifications of the MIT-E3 input parameters which incorporates properties of natural
BBC would improve the numerical capabilities even further.
process played a limited role in defining the vertical movements at ISS-4, especially on the North side. Unfortunately, there were no piezometers placed within the clay stratum outside the excavation. These instruments could have revealed the extent of disturbance caused by tieback installation and subsequent influence on the consolidation of soil.

Despite the difficulties in capturing the behavior of non-BBC materials and the tieback installation effects in the numerical model, this South Boston case demonstrated that reliable and reasonable predictions of the diaphragm wall deflections, lateral movements at and below the toe of the sheetpiling wall, and surface settlements at points beyond the influence of the tiebacks are possible with the current finite element modeling techniques.
4.3 THE TRANSITWAY PROJECT

The proposed Transitway is an addition to MBTA's public transit network. This project consists of a one-mile long two-lane subway tunnel with three stations connecting South Station in downtown Boston, the new Federal Courthouse, and the Would Trade Center in South Boston (Figure 4.1.1). The tunnel and station sections are currently designed to accommodate electric buses with provisions for future conversion to light rail. The section of interest covers Stations 106+00 to 124+50 (see Figure 4.3.1) and includes the proposed Courthouse Station and a small portion of the underground roadway. This section describes numerical analyses used to estimate and evaluate the performance of the proposed braced excavations at critical sections of the alignment.

4.3.1 Site Description

Within the section of interest, four typical cross-sections (Platform, Mezzanine, Transition, and West Tunnel) have been considered. These four cross-sections have excavation widths ranging from 9.8-m to 33.5-m and maximum excavation depths ranging from 17.7-m to 18.6-m (Table 4.3.1). The Platform and the Mezzanine sections describe the Courthouse Station. The Transition section describes the transition connecting the station and the underground roadway. The West Tunnel section represent the typical excavation for the underground roadway. Sections 4.3.2 and 4.3.3 describe the general conditions, proposed construction sequence, and support structures. The finite element model and the corresponding input parameters are discussed in Section 4.3.4. The numerical results and their analyses of results for the four sections are presented in Sections 4.3.5 to 4.3.8. Evaluation of the
prediction capabilities and limitations of the numerical analyses are discussed in Section 4.3.9.

Table 4.3.1 also summarizes the proposed lateral earth support conditions for each of the four sections. The preliminary design calls for reinforced concrete diaphragm wall (0.9-m thick) to support 18-m deep excavations at the Platform, Transition, and the West Tunnel sections. The Mezzanine Section has two levels of excavations, the upper level (width 33.5-m and depth 10.4-m) will be supported by outer diaphragm wall (DW) while inner sheetpile wall (SPW - section PZ38) will support the inner excavation (width 18.3-m) to the final excavation depth of 17.7-m.

All of the support walls will be braced by 1.9-cm thick, 76.2-cm diameter steel pipe struts\(^{112}\) with a design load of 4,450-kN. The calculations assume a horizontal strut spacing of 6.7-m for all four sections while the proposed vertical spacing for each section varies between 3 to 4-m as summarized in Table 4.3.1. The proposed design assumes that there is no prestress applied in the struts at the time of installation (i.e. passive strut, Figure 2.4.4).

The performance of the proposed braced excavations for each typical cross-section is evaluated based on the following measures:

1. Ground Deformations
   a. Lateral wall deflection
   b. Surface settlement
2. Stresses on Structural Members
   a. Bending moments in the support wall
   b. Axial forces in the bracing struts
   c. Lateral pressures imposed on the support walls

In addition to the evaluation of the proposed braced excavations, the numerical analysis also examined how wall embedment, vertical strut

\(^{112}\) Other properties for the pipe strut: \( A = 69 \text{ in}^2 = 445.2 \text{ cm}^2 \), \( E = 29,000 \text{ ksi} = 2.0 \times 10^6 \text{ MN/m}^2 \), horizontal spacing = \( S = 22.0 \text{-ft} = 6.7 \text{-m} \).
spacing, partial drainage, and the uncertainties in the Boston Blue Clay (BBC) strength profile influence the excavation behavior.

4.3.2 Soil Profile

The soil profile and soil properties at the Transitway site are based on borings obtained for this project [PBQ&D, 1995] as well as site investigation data from the South Boston Special Test Program [Estabrook, 1991; Haley & Aldrich Inc. 1993] located on the east end of the section (near Station 131+00) as shown in Figure 4.3.1.

Figure 4.3.7 shows the average soil profile at this site with the ground surface at El. 112.0-ft, and the bottom of the Boston Blue Clay/top of glacial till at El. -25.0-ft. This profile comprises four main soil strata: 1) 1.8-m of miscellaneous fill [El. 112 to 106-ft]; 2) 6.7-m of hydraulic/cohesive fill [El. 106 to 84-ft]; 3) 3.7-m of silty sand [El. 84 to 72-ft]; and 4) 29.6-m of Boston Blue Clay [El. 72 to -25-ft]. Groundwater condition is similar to the South Boston site (Figure 4.2.6) with the groundwater table at El. 106-ft for the upper aquifer, while the piezometric elevation in the underlying till is at El. 100.0-ft. Figure 4.3.7 shows the in-situ vertical stresses and pore pressures based on unit weights listed in Table 4.1.2 and assuming that the (6.0-ft) piezometric head loss occurs within the BBC stratum. The soil properties for the four layers are summarized in Sections 4.3.2.1 and 4.3.2.2.

4.3.2.1 Soil Properties for the Upper Soil Strata

The strength and deformation properties for the miscellaneous fill, cohesive fill, and the silty sand are estimated based on SPT-N blowcounts and additional data compiled by Whelan [1995] from other sites in South Boston.
The engineering properties for these upper soil strata are summarized in Table 4.3.2.

4.3.2.2 Properties of the Boston Blue Clay

The principal source of data on stress history and strength properties of the Boston Blue Clay at the Transitway site are from an extensive program of laboratory tests performed at the South Boston Special Test Site (SBSTS) [Haley and Aldrich, 1993]. Figure 4.3.1 shows that this special test site is located approximately 150-m to the South of the Courthouse Station, just beyond the end of the section of interest. A second important source of data are results of field vane tests performed in a series of six borings as a part of the site investigation program for the Transitway project (PBQ&D, 1995).

Figure 4.3.2 summarizes the stress history profile at the SBSTS based on results of more than 70 laboratory 1-D (conventional oedometer and Constant Rate of Strain, CRS) consolidation tests. The deposit can be subdivided into a 13.7-m thick main clay unit which is very lightly overconsolidated with OCR \( \approx 1 \) to 1.3; and a upper 15.8-m thick clay crust where the OCR increases with elevation. The figure also shows a best estimate of the pre-consolidation pressures ("Original Profile") which has been used in subsequent analyses. Figure 4.3.3 shows the corresponding MIT-E3 predictions of the undrained shear strength for triaxial compression (\( s_{u TC} \)), triaxial extension (\( s_{u TE} \)), and direct simple shear (\( s_{u DSS} \)) modes, based on the input parameters listed in Table 3.1.1. The computed strength profiles (i.e. MIT-E3 model with original stress history profile) can be evaluated by comparison with results of laboratory strength tests from the SBSTS. Figure 4.3.4 summarizes the results of (a) undrained triaxial compression test (\( C_{k UC} \)) in which specimens are reconsolidated to their estimated in-situ stress state; (b) undrained triaxial
(K₀UC), and direct simple shear strength (K₀UDS) using SHANSEP consolidation procedures and normalized strength properties (after Ladd and Foot, 1974)¹¹³. The results show good agreement between triaxial compression strengths from recompression and SHANSEP tests (sᵤTC). However, the MIT-E3 model (with original σ'ₚ profile), overestimates these laboratory strengths by 15 to 25%. Similar differences can also been seen in the undrained direct simple shear strengths, sᵤDSS. Unfortunately, there is insufficient data to evaluate the undrained triaxial extension strength predictions.

Further validation of model strengths with the field vane data is shown in Figure 4.3.5. There is much more scatter in the field vane data, especially in the lower clay, where sᵤFV = 30 to 10kPa (at depth 37-m). However, the results again suggest that the original stress history profile leads to a overestimation of the undrained shear strength of BBC at the Transitway site. Based on the comparisons in Figures 4.3.4 and 4.3.5, a Revised Stress history profile has been developed by matching MIT-E3 predictions of undrained shear strength (sᵤDSS) with the field vane data given in Figure 4.3.5. Figure 4.3.2 shows that this revised stress history profile underestimates σ'ₚ values measured in the crust, and assumes OCR = 1.0 at depths greater than 28-m.

In principle, the measured field vane strengths, sᵤFV, should match the (SHANSEP) sᵤDSS profile. However, comparison of results shows that although there is reasonable agreement in the crust, the sᵤFV data are typically smaller than sᵤDSS in the lower clay (below 27-m)¹¹⁴.

¹¹³ Normalized strength parameters are interpreted from results of tests performed at different OCR's using the following equation (SHANSEP): sᵤ/σ'ᵥc = S(OCR)ᵐ where S and m are empirical constants.
¹¹⁴ The very low measurements of strength (sᵤFV < 1 ksf) below El. 20-ft are considered questionable data as reported by Parsons Brinckerhoff Quade & Douglas, Inc.
Figure 4.3.6 confirms that there is very good agreement between the undrained shear strengths computed by MIT-E3 with the Revised Stress history profile and the laboratory data from the SBSTS. The inconsistency between stress history ($\sigma'_P$) and strength data ($s_u$) is due to differences in the normalized strength properties measured for natural BBC (at the SBSTS) compared to those obtained on resedimented BBC\textsuperscript{115}. This issue can be resolved by revising the input properties of the MIT-E3 model (Table 3.1.1) using data for natural BBC. Unfortunately, this task is beyond the scope of the present thesis.

In the current analyses, uncertainties in the undrained shear strength are estimated by comparing predictions of excavation performance using the Original and Revised stress history profiles. As demonstrated in Section 3.3, the difference in stress history of the lower clay has an important impact on the behavior of deep ($H > 10$-m) and therefore, the consideration of the strength profile is very significant for this case study.

### 4.3.3 Proposed Construction Sequence

The proposed construction sequence at the Platform, Mezzanine, Transition, and West Tunnel sections are illustrated in Figures 4.3.8 to 4.3.13. As summarized in Table 4.3.1, three variations in wall lengths and the vertical strut spacings are considered for the Platform Section (P1, P2, and P3), while a single pre-defined excavation geometry is assumed at the Mezzanine, Transition, and West Tunnel Sections (M1, T1, and W1).

The three excavation geometries considered at the Platform Section are as follows:

\textsuperscript{115} The MIT-E3 input parameters in Table 3.1.1 were based on laboratory data obtained for resedimented BBC NOT natural deposit of BBC (Whittle et al., 1994).
1) P1: L = 26.8-m (toe of wall at El. 24-ft); h = 3-m
2) P2: L = 26.8-m (toe of wall at El. 24-ft); h = 4-m
3) P3: L = 23.8-m (toe of wall at El. 34-ft); h = 4-m

The Mezzanine (M1), Transition (T1), and West Tunnel (T1) Sections all assume a vertical strut spacing of h = 4-m [h = 13-ft] with wall lengths listed in Table 4.3.1.

Similar excavation sequences are proposed at all four sections: (a) initial 2.1-m of unsupported excavation (to El. 105-ft); which is followed by a sequence of (b) additional tiers of excavation and installation of struts with a duration of 45 days/tier of excavation. The excavation of soil and installation of struts continues until the proposed final excavation grade is reached at which time, the drainage period is increased to 135 days.

4.3.4 Finite Element Model and Input Parameters

The numerical models used for this Transitway project are based on the site description, soil profile, and the proposed construction sequence and support structures described in Sections 4.3.1 to 4.3.3. The finite element mesh, element type, material properties, and analyses performed are summarized in the next 4 sub-sections.

4.3.4.1 Finite Element Mesh

Figure 4.3.14 shows the finite element (FE) mesh for the P3 analysis. In general, the numerical analyses assume that the excavations at Transitway remain symmetric and in plane strain throughout the excavation period thus the finite element mesh consists of half of the excavation with plane strain conditions. The base of the mesh is at the base of the BBC stratum / top of glacial deposit with displacement constraints imposed in both the x- and y-
directions (see Figure 4.3.14). Laterally, the mesh extends 210.0-m [690-ft] from the support wall and no movement in the x-direction is imposed at the far right extent as well as at the centerline.

The initial stresses and pore pressures in the numerical model reflect the unit weights of the soil and piezometer data measured in the fill (Table 4.3.2 and Figure 4.3.7). Partially drained analyses (PD) assume that there are no pumping activities at the site, and hence pore pressures remain constant in the underlying till/bedrock. As shown in Figure 4.3.14a, the FE mesh also accounts for the excavation tiers, strut locations, and the location of the support wall.

4.3.4.2 Element Type

Three types of elements are used in the FE mesh for the Transitway analyses: 1) 8-node solid isoparametric elements for modeling the structural wall and dry soil; 2) 1-D spring elements for modeling the cross-lot bracing; and 3) 8/4 mixed quadrilateral element with eight displacement nodes and four pore pressure nodes for modeling soil.

4.3.4.3 Material Model and Input Properties

Both the diaphragm wall and the sheetpile wall are modeled using two columns of elastic solid elements with input properties matching the bending and axial stiffness of the actual sections. Table 4.3.3 summarizes properties used in the analyses. The FE model also assumes that the walls are wished-in-placed; therefore no disturbance in the surrounding soil at the initial step. Since the finite element model does not include interface elements, any slippage at the soil-wall interface is completely controlled by the shear resistance of the soil adjacent to the wall.
One-dimensional elastic spring elements are used to model the struts. The equivalent spring constant is obtained based on the proposed strut size, elastic modulus, vertical and horizontal spacing, and excavation width. The input parameters for the cross-lot bracing are also summarized in Table 4.3.3.

The (dry) miscellaneous fill and (saturated) sand layers are modeled using a simple elasto-plastic model with a Drucker-Prager failure criterion and a non-associated flow role (EP-DP model). Input parameters for these two layers are listed in Table 4.3.2 and include 1) the in-situ $K_0$ stress ratio, 2) the friction angle for plane strain shearing, $\phi'$, 3) the normalized elastic shear modulus, $G/\sigma'_{vo}$, and 4) the Poisson's ratio, $\nu'$.

The cohesive fill has a much lower permeability than either the miscellaneous fill or underlying silty-sand. The analyses assume that this layer will experience only partial drainage during the time frame of the proposed excavation. Hence, the cohesive fill is modeled as an Elastic-Perfectly Plastic material with the undrained shear strength and stiffness proportional to the in-situ vertical effective stress ($S_{uPS}/\sigma'_{vo} = 0.25, G/S_{uPS} = 100$). The input parameters for the cohesive fill is also included in Table 4.3.2.

The behavior of the Boston Blue Clay layer is modeled by the MIT-E3 soil model with the material parameters in Table 3.1.1 and two stress history profiles, Original and Revised Profiles\textsuperscript{116}, shown in Figure 4.3.2. A constant average permeability of $4.9 \times 10^{-8}$ cm/sec [$k = 1.6 \times 10^{-9}$ ft/sec] is assumed for the BBC based on extensive laboratory tests performed at the MIT test site in Saugus [Ghantous, 1982]. This property is typical of the lower BBC, but may underestimate the amount of drainage occurring within the overconsolidated crust (see Section 4.2).

\textsuperscript{116} The FE analyses use a constant OCR within a given finite element; hence, the pre-consolidation pressures follow a step function (which reflect the mesh resolution) with the average values shown in the figures.
4.3.4.4 Summary of Numerical Analyses

Results from a total of 12 numerical analyses are presented in the next 4 sections. Table 4.3.4 summarizes the 12 analyses performed. The first three analyses (P1-O-UD, P2-O-UD, and P3-O-UD) examine the three possible dimensions of the Platform Section using the Original Profile (O) and assuming undrained conditions\textsuperscript{117} (UD). The goal of these three analyses is to examine the impact of changes in the wall length \((L = 26.8\text{-m vs. } 23.8\text{-m})\) and strut spacing \((h = 3.0\text{-m vs. } 4.0\text{-m})\) for an excavation depth of \(H = 17.7\text{-m}\).

Three other analyses (P1-O-PD, P3-O-PD, and P3-R-PD) also predict the behavior at the Platform Section. The first two, P1-O-PD and P3-O-PD, examine the impact of partial drainage for excavations P1 and P2 with the Original Profile, allowing partial drainage (PD) (45 days per excavation step and 135 days at the final step). The last analysis at the Platform section, P3-R-PD, examines the impact of the using a lower undrained strength profile. This a partially drained analysis uses the Revised Profile (R) to define the initial stress history within BBC.

Each of the Mezzanine, Transition, and the West Tunnel Sections consists of two partially drained analyses: the first one with the Original Profile (M1-O-PD, T1-O-PD, and W1-O-PD); and the second analysis with the Revised Profile (M1-R-PD, T1-R-PD, and W1-R-PD). The resulting wall deflections, surface settlements, surface horizontal movements, moment distribution in the wall, and strut loads for each of the four typical cross-sections are summarized in the next four sections.

\textsuperscript{117} This is accomplished by introducing an artificially small time frame in the numerical analysis.
4.3.5 Numerical Results for Platform Section

The evaluation of the Platform Section consists of six numerical analyses (see Table 4.3.4) with different support geometries, drainage condition, or strength profile. The first three (P1-O-UD, P2-O-UD, and P3-O-UD) examine the impact of changes in vertical strut spacing and wall length; the results are presented in Section 4.3.5.1. Section 4.3.5.2 examines the impact of partial drainage by comparing results obtained from P1-O-UD vs. P1-O-PD and P3-O-UD vs. P3-O-PD. The effect of strength profile is evaluated in Section 4.3.5.3 based on results from P3-O-PD and P3-R-PD analyses.

4.3.5.1 Effect of Strut Spacing and Wall Length

The first series of analyses for the Platform Section consists of P1-O-UD, P2-O-UD, and P3-O-UD which evaluate the effect of the vertical strut spacing and wall length. The construction sequence and the support structure for these three excavation geometries (P1, P2, and P3) are illustrated in Figures 4.3.8 to 4.3.10. As indicated earlier, all three analyses use the Original Profile as the stress history for the BBC layer and undrained conditions throughout the analyses.

Figures 4.3.15 to 4.3.17 show the wall deflection, surface settlement, surface horizontal displacements, and the moment distribution obtained from the P1-O-UD analysis. The proposed excavation for P1 consists of 6 stages (Steps 1 to 6) with a final excavation depth of 17.1-m [to El. 54-ft]. Steps 7 and 8\(^{118}\) are excavation of two additional tiers for the purpose of evaluating the basal stability. The deflected wall shapes (Figure 4.3.15) display the typical phases reported in Chapter 3: initial cantilever movement (Step 1), followed

---

\(^{118}\) Steps 7 and 8 [P1-O-UD] consist of excavations to depths of 18.9 and 23.2-m [El. 48-ft and 34-ft] and two additional levels of struts at depths of 16.5 and 18.9-m [El. 56-ft and 48-ft].
by "bulging" (Steps 5 and 6), and ultimately toe "kick-out" (Step 7). At the final proposed grade (Step 6), the maximum wall deflection occurs near the toe of the wall with a maximum inward movement $\delta_{w(\text{max})} = 6.3$-cm. The corresponding surface settlements and surface horizontal movements for P1-O-UD are shown in Figure 4.3.16. Typical settlement troughs are observed with outward migration of the locus of maximum settlement until Step 7, when the toe kick out occurs. At the final invert (Step 6), the maximum settlement is $\delta_{v(\text{max})} = 3.4$-cm at $x = 13.4$-m from the wall. The bending moment in the diaphragm wall is shown in Figure 4.3.17. The predicted magnitudes of moments in the wall are below the allowable moment of 1.2 MN-m/m [265 kips-ft/ft] for the 0.9-m concrete diaphragm wall\textsuperscript{119}.

Results from the P2-O-UD analysis are reported in Figures 4.3.18 to 4.3.20. As indicated in Figure 4.3.9, the proposed excavation for P2 has the same wall length, as P1 but with a larger (4-m instead of 3-m) vertical strut spacing and larger lift off each step of excavation; consequently, the final grade is reached at Step 5. As in the proceeding case, two additional tiers of excavations were included (Steps 6 and 7\textsuperscript{120}) for the purpose of evaluating basal stability. The wall deflections, surface settlements, surface horizontal displacements, and the moment distributions display similar characteristics as observed for the P1-O-UD analysis. In fact, the displacements at final grade (Step 5) are virtually identical to the displacements predicted for the P1-O-UD case.

Figure 4.3.21 summarizes the maximum displacements observed for the P1-O-UD and the P2-O-UD cases (vertical strut spacing of 3-m vs. 4-m) as a function of excavation depth. As indicated earlier, the difference of 0.9-m [3-

\textsuperscript{119} Note that the maximum moments occur at two locations: one ("negative moment") above the lowest level of support and the other ("positive moment") below the lowest support.
\textsuperscript{120} Steps 6 and 7 [P2-O-UD] consist of excavations to depths of 21.3 and 23.8-m [El. 40 and 32-ft] and two additional levels of struts at depths of 17.1 and 21.3-m [El. 54 and 40-ft]
ft] in vertical strut spacing has limited impact on the soil movements. In terms of forces on the structural members, the vertical support spacing affects the bending moment diagrams, especially above the excavation grade; However, it has minimal influence on the maximum bending moments above and below the lowest support, as shown in Figure 4.3.22. The predicted moments were all within the allowable moment for the entire excavation.

Figure 4.3.23 shows the predicted forces in the struts for both analyses. As expected, the increase in vertical strut spacing increases the forces in the struts since fewer struts are used. In the case of P1-O-UD, forces in all 5 struts were within the design load of 657-kN/m [45-kips/ft] at the proposed final grade. P2-O-UD, on the other hand, predicts a force of 690-kN/m [48-kips/ft] in Strut 3 at the proposed final excavation stage while forces in the other 3 struts are below the allowable load. Therefore, the 0.9-m difference in vertical strut spacing has negligible impact on the soil movements but it does influence the strut loads.

P3-O-UD, the third analysis of the Platform Section, consists of a shorter wall (3-m shorter) than P1 and P2 and the same strut spacing of 4-m as P2. The wall deflections, surface settlements, surface horizontal movements, and moment distributions are shown in Figures 4.3.24 to 4.3.26. The characteristic behaviors observed for the P1-O-UD and P2-O-UD cases are also observed in this P3-O-UD case.

The effect of wall length can be evaluated by comparing results from this P3-O-UD case with P2-O-UD. Figures 4.3.27 to 4.3.29 summarizes the maximum soil displacements, maximum bending moments in the diaphragm wall, and the forces in the four levels of struts for these two cases. The lateral wall deflections, surface displacements, bending moments, and strut forces for both analyses are virtually identical. These results show that
the additional wall embedment of 3.0-m [10-ft] is not effective in reducing the
ground movements for this particular soil profile (cf. Sections 3.2 and 3.3 on
parametric analysis of wall length and clay history profile).

This first set of analyses examined the impact of changes in vertical strut
spacing (by 0.9-m) and wall length (by 3-m). Results of three undrained
analyses with the Original Profile showed that these changes have a negligible
impact on the soil movements and moments in the wall; the largest observed
impact is in the increase in strut forces when the number of struts is reduced
due to the increase in the vertical strut spacing.

4.3.5.2 Effect of Partial Drainage

A complete analysis of partial drainage of pore pressures requires
detailed knowledge of the permeability properties of the soil layers, location
of drainage horizons, the time history of construction, and other related
activities such as deep pumping. The following assumptions have been
made in the current analyses: 1) average permeability values are used for each
soil layer, 2) there is no deep pumping at the site, 3) the underlying glacial till
is rather permeable such that a constant hydraulic head is sustained at the
clay/till interface, 4) the excavation activities do not influence the pore
pressures 210-m from the excavation, and 5) the calculations assume 45 days
per excavation step and 135 days for the final step (proposed final grade).

Figures 4.3.30 to 4.3.32 show the wall deflection, surface displacements,
and moments in the wall for the partially drained analysis of the P1 support
gallery with the Original Profile (P1-O-PD). Comparisons of maximum
displacements, maximum bending moments, and forces in struts from the
undrained and partially drained analyses for P1 (P1-O-UD and P1-O-PD) are
shown in Figures 4.3.33 to 4.3.35. In terms of wall deflections, partial drainage
causes a 1.8-cm [0.7-in] reduction in the initial cantilever wall deflections (Step 1, Figures 4.3.15, 4.3.30, and 4.3.33). The deflected mode shapes of the wall and incremental movements during subsequent phases of the excavation are almost identical. The differences observed in the initial cantilever stage persist through the remainder of the excavation. These results show that partial drainage effects have a limited impact on the behavior of BBC, but control ground movements within the upper fill and sand materials. Though this initial cantilever movement is rather significant at early stages, as the excavation approaches the final grade (Step 6), maximum wall deflection is expected to occur at the toe of the wall under both undrained and partially drained conditions. Similar to the wall deflections, partial drainage also causes significant reductions in the surface settlement and surface horizontal displacements (Figures 4.3.16, 4.3.31, and 4.3.33).

Figures 4.3.34 and 4.3.35 summarize the impact of partial drainage on loads on structural elements: moments in the wall and forces in the struts (also see Figures 4.3.17 and 4.3.32). In general, partial drainage yield slight reductions the predicted bending moment in the wall and the strut forces.

A partially drained analysis was also performed for the P3 support condition with the Original Profile (P3-O-PD). The results are reported in Figures 4.3.36 to 4.3.38. Comparisons of maximum displacements, maximum bending moments, and strut forces from P3-O-UD and P3-O-PD cases are shown in Figures 4.3.39 to 4.3.41. Similar results as those for P1 are also observed for the P3 support geometry. Partial drainage also accounts for a 1.8-cm reduction in the initial cantilever movement in the wall, as well as significant reductions in surface settlement and horizontal displacements. A
reduction in the strut forces below the design load was also observed in the partially drained case.

The two partially drained analyses, P1-O-PD and P3-O-PD, showed that partial drainage reduces the initial cantilever wall movement by almost 2-cm (close to 45%) and eventually retains this difference in maximum wall deflections in the subsequent stages of excavations. Partial drainage also accounts for the reductions in surface settlements and surface horizontal movements. In terms of loads on structural elements, the strut forces in the lower levels are generally smaller under partial drainage conditions. The analyses also indicate that the proposed construction for the Platform Section should be stable over a construction time period of over 300 days (360 days for P1-O-PD and 315 days for P3-O-PD).

4.3.5.3 Effect of Clay Strength Properties

The effects of decreases in the undrained shear strengths within the BBC are evaluated by comparing predictions for the P3 support geometry with the Original and the Revised stress history profiles (P3-O-PD and P3-R-PD) under partially drained conditions. The resulting wall deflections, surface settlements, surface horizontal displacements, and moments in the wall are shown in Figures 4.3.42 to 4.3.44. Summary plots of maximum displacements, maximum bending moments, and strut forces are included in Figures 4.3.45 to 4.3.47.

Changes in the stress history profile with the BBC layer have a limited impact on shallow excavations (H < 10.0-m). Figure 4.3.42 shows the wall deflections predicted using both the Original and Revised Profiles. At Step 1, the initial cantilever wall movements are unaffected by the properties in the underlying clay stratum. As the excavation progresses, the differences in the
toe movements become more dramatic. At the proposed final grade (Step 5), the P3-R-PD analysis (with the Revised Profile) predicts a maximum toe movement of 8.8-cm, which is almost twice as large as that from the Original Profile ($\delta_{w(\text{max})} = 4.8$-cm). The deflected mode shape (P3-R-PD) at final grade also indicates a region below the wall where the soil is failing, therefore suggesting that the full lateral resistance of the embedded wall has been reached.

Surface settlements and surface horizontal displacements also show similar trends as the wall displacements: virtually identical movements in the initial step, and at the final stage (Step 5), the Revised Profile (P3-R-PD) reported displacements nearly twice those of the Original Profile (P3-O-PD). However, the stress history profile has little influence on the distribution of the surface settlements and the horizontal displacements. Both analyses have the same locations of maximum displacements and negligible displacements beyond 90-m.

The stress history profiles also influence the loads on the structural elements; however, the impact is not as dramatic as the differences observed in the soil movements. The Revised Profile analysis (P3-R-PD) generates slightly higher bending moments than the Original Profile analysis (P3-O-PD) (see Figures 4.3.38, 4.3.44, and 4.3.46). Nevertheless, the projected moments are still within the allowable range $M_{\text{max}} < 1.2 \text{ MN-m/m}$ [265 kips-ft/ft]. With the exception of the top strut (Strut 1), higher strut forces are also projected by the Revised Profile analysis for Struts 2, 3, and 4 (see Figure 4.3.47). The Revised Profile analysis predicts the force in Strut 3 (El. 79-ft) will exceed the allowable load of 657-kN/m at the end of the 315 construction days; however, the most noticeable difference is the force in Strut 4 (lowest level at El. 66-ft). The Revised Profile analysis predicts a continued increase in the
strut force during partial drainage after reaching the proposed final grade. After 135 days of partial drainage, the force in Strut 4 reaches 720-kN/m [50-kips/ft]. In contrast, the load in Strut 4 is projected to remain stable and below the allowable load for the Original stress history profile.

It can be concluded that, changes in the stress history profile have a dramatic impact on soil displacements, especially for excavation depths greater than 10-m. In the case of the Platform Section, reductions in the stress history profile can cause the projected displacements to double at the final excavation stage [H = 17.1-m]. Changes in the stress history profile also cause increases in the bending moments in the wall and strut forces. For the Platform Section, the forces in the lower two levels of struts are of concern since the Revised Profile analysis (P3-R-PD) projects loads which exceed the design capacity of the streets.

4.3.6 Numerical Results for Mezzanine Section

The proposed layout for the Mezzanine Section is shown in Figure 4.3.48. The proposed excavation sequence is illustrated in Figure 4.3.11. As shown in both figures, the Mezzanine section comprises two sets of support walls. The 17.1-m long reinforced concrete diaphragm outer walls support the upper 33.5-m wide concourse level which will be excavated to a depth of 10-m [to El. 77-ft]. The inner sheetpile walls (PZ-38), which extend to El. 44-ft, are designed to support a 18.3-m wide excavation to El. 54-ft. Four levels of struts\textsuperscript{121} with vertical spacing of 4.0-m are proposed for this section. The top two struts (Struts 1 and 2) support the outer diaphragm walls; the lower two levels of struts (Struts 3 and 4) support the inner sheetpile walls. With this

\textsuperscript{121} The same strut sections as used in the Platform Section are used at the Mezzanine Section as well as the Transition and West Tunnel Sections.
single geometry for the Mezzanine Section, two partially drained analyses have been performed: one using the Original Profile (M1-O-PD) and the other using the Revised Profile (M1-R-PD). Results from these two analyses are shown in Figures 4.3.49 to 4.3.54.

The projected wall displacements from the two analyses for the two sets of walls are shown in Figure 4.3.49. The wall deflections for the sheetpile wall (SPW) are shown on the left and the results for the diaphragm wall (DW) are shown on the right. Horizontal movements of the diaphragm wall for the first three stages (Steps 1 to 3) are very similar to the results from the P3 analyses (P3-O-PD and P3-R-PD) since both sections involve a 33.5-m wide excavation to El. 77-ft. Hence, the shorter support wall (6.1-m [20-ft] shorter) at the Mezzanine Section has limited impact on the wall deflections. Excavation Steps 4 and 5\textsuperscript{122} involve a narrower excavation; therefore, the deflection of the diaphragm wall for the Mezzanine Section is smaller than projected for the Platform Section. At the end of the construction period, the toe the diaphragm wall is projected to move 3.8-cm [1.5-in] to 6.6-cm [2.6-in] depending on the stress history profile. The sheetpile remains unsupported until Step 4 when Strut 3 is installed at El. 79-ft. The maximum horizontal movements at the sheetpile wall at the end of the construction period are slightly larger than the movements at the diaphragm wall (see Figures 4.3.49 and 4.3.52).

The surface settlements and surface horizontal displacements are reported in Figure 4.3.50. The maximum surface displacements as a function of excavation depth are plotted as Figure 4.3.52. The surface displacements for the first three steps are virtually identical to the Platform Section since equal

\textsuperscript{122} Note that the discontinuity observed at a depth of 26.2-m [El. 24-ft] in M1-R-PD results also mark the top of normally consolidated (OCR = 1) BBC in the Revised Profile. The Original Profile has a minimum OCR = 1.15 below El. 16'.

Page 445
amounts of soil were excavation during the first three stages. For the last two steps (Steps 4 and 5), smaller displacements are projected at the Mezzanine Section since the width of the excavation is reduced to 18.3-m. However, the location of maximum settlement is unaffected by the changes in excavation width and still occurs at $x = 15.2$-m from the diaphragm wall at the final construction stage.

Projected loads on the structural elements are shown in Figures 4.3.51, 4.3.53, and 4.3.54. Figures 4.3.51 and 4.3.53 show the moment distributions and the maximum bending moments as a function of excavation depth for the diaphragm and sheetpile walls ($M_y = 1.2$ MN-m/m for diaphragm wall and $M_y = 430$ mN-m/m). Both numerical analyses predict moments that are well below the allowable level for both walls. The projected strut loads, shown in Figure 4.3.54, are also below the recommended level.

The analysis of the Mezzanine Section involves two partially drained numerical simulations: one using the Original Profile (M1-O-PD) and the other assuming the Revised Profile (M1-R-PD). The differences due to this change in stress history profile observed in the analysis of the Platform Section are also present in the Mezzanine Section. The Mezzanine Section displays similar behavior as the Platform Section but with smaller displacements in the last two stages due to the reduced excavation width.

4.3.7 Numerical Results for Transition Section

The proposed design and construction sequence for the Transition Section are shown in Figures 4.3.55 and 4.3.12, respectively. The proposed wall length (26.2-m), support conditions (4-m), and the excavation sequence for the Transition Section are identical to the Platform Section P2 Geometry.
The only difference is that the Transition Section has an excavation width of 16.5-m, which is approximately half as wide as the Platform Section.

The analysis of the Transition Section involved two numerical simulations, referred to as T1-O-PD and T1-R-PD. Both analyses are partially drained with the only difference being the stress history within the BBC stratum (T1-O-PD - original and T1-R-PD Revised stress history profiles). The results of these two analyses are shown in Figures 4.3.56 to 60.

Figure 4.3.56 shows the wall deflections, as well as the moments in the wall at all proposed excavation steps for both analyses. The surface settlements and surface horizontal displacements are reported in Figure 4.3.57. The maximum displacements as a function of excavation depth are summarized in Figure 4.3.58. Maximum bending moments in the wall as a function of excavation depth are shown as Figure 4.3.59. The strut loads are shown in Figure 4.3.60.

In general, the Transition Section displays very similar behavior to the Platform Section. The reduction in excavation width has a negligible impact on the distribution of the displacements. However, the reduced excavation width does cause the magnitude of displacements to be smaller than those predicted for the Platform Section. The maximum wall displacement at the final stage range from 3.6-cm to 6.5-cm for the two analyses. In terms of loads in the structural elements, the moments in the diaphragm wall are projected to fall well within the recommended range. The projected strut forces, however, raise some concern. The top two levels of struts (Strut 1 and 2) report forces less than 440-kN/m whereas forces exceeding the design load capacity (657-kN/m) are predicted for Struts 3 and 4.
4.3.8 Numerical Results for West Tunnel Section

Figure 4.3.61 shows the proposed design for the West Tunnel Section. The proposed construction sequence is illustrated in Figure 4.3.13. The support conditions and excavations sequence for the West Tunnel are very similar to the Transition Section. Both sections require four levels of struts spaced vertically at 4-m apart, but the West Tunnel Section has the following differences: 1) a slightly shorter concrete diaphragm wall (0.6-m = 2-ft shorter), 2) an narrower excavation width (BWT = 9.8-m = 32-ft); and 3) a slightly deeper final grade (0.9-m deeper to El. 51-ft.

The evaluation of the West Tunnel Section also involves two partially drained numerical analyses (W1-O-PD -Original and W1-R-PD, Revised stress history profiles). Results from both analyses are reported in Figures 4.3.62 to 4.3.66.

The projected wall deflections and the moments in the wall for both analyses are shown are Figure 4.3.62. The surface settlements and surface horizontal displacements are included in Figure 4.3.63. Figures 4.3.64 and 4.3.65 are summary figures of maximum displacements, and maximum bending moments in the wall as a function of excavation depth. As anticipated, the behavior of the proposed West Tunnel excavation is very similar to the Transition Section. Though the most apparent difference is the somewhat lower magnitude of displacements due to the reduction in the excavation width, there are other observations that should be noted. As discussed in Section 3.2, for very narrow excavations, the maximum settlement is located closer to the excavation compared to wider excavations. In the case of the West Tunnel Section, the maximum settlement at the final stage occurs at approximately 12.8-m from the diaphragm compared to 15-m for the Transition Section. The final bending moments in the diaphragm
wall are also higher in the West Tunnel. This increase in the bending moment can be attributed to the large strut force predicted for this section. Nevertheless, the projected moments at all stages of excavation are within the allowable range ($M_{y(DW)} = 1.2$ MN-m/m and $M_{y(SPW)} = 433$ kN-m/m). The predicted strut forces are reported in Figure 4.3.66. As shown in this figure, forces beyond the allowable level are expected at the lower two levels (Struts 3 and 4). In fact, the force in Strut 4 may reach a level which is twice the design load at the end of the construction period. These predictions of strut forces suggest that the current proposed bracing systems for both the West Tunnel Section and the Transition Section need to be modified.

4.3.9 Evaluation of the Proposed Excavation for Transitway Project

The proposed braced excavations for the Transitway Project consist of four typical sections (Figures 4.3.8 to 4.3.13) with excavation widths ranging from 9.8-m to 33.5-m and slight variations in the support structures for excavation depths of 17.7-m to 18.6-m. The performance of these four sections is evaluated using 12 numerical analyses summarized in Table 4.3.4. These 12 analyses evaluate the proposed design for the four sections, and also examine the impact of variations in wall length, vertical support spacing, partial drainage effects, and reductions in the strength profile.

Sections 4.3.5.1 to 4.3.5.3 discuss the impact of wall length, vertical support spacing, partial drainage effects, and differences in the stress history profile on the performance of the excavation. The maximum wall deflections ($\delta_{w(max)}$), surface settlements ($\delta_{v(max)}$), and surface horizontal displacements ($\delta_{h(max)}$) are summarized in Table 4.3.5. The evaluation of the proposed structural supports is summarized in Table 4.3.6. In summary, a 3-m increase in wall length (P2-O-UD vs. P3-O-UD) and a 0.9-m increase in vertical strut
spacing (P1-O-UD vs. P2-O-UD) have negligible impact on the soil movements and the moments in the wall. The strut forces, however, are influenced by the strut spacing since the total number of struts is reduced with increased spacing. Partial drainage (P1-O-UD vs. P1-O-PD and P3-O-UD vs. P3-O-PD) causes a significant reduction in the wall and surface displacements. The reductions in the stress history profile (and strength profile) cause significant increases in the soil movements. The displacements predicted for the Original Profile as the stress history profile are only about half of the displacements for the Revised Profile (see Table 4.3.5, P3-O-PD vs. P3-R-PD, M1-O-PD vs. M1-R-PD, T1-O-PD vs. T1-R-PD, and W1-O-PD vs. W1-R-PD).

Overall, the behaviors of the Platform, Mezzanine, Transition, and the West Tunnel sections are very similar and considered stable in terms of basal stability and strength of the wall for the specified construction period. However, if the Revised stress history profile is more appropriate for this site, special considerations, such as excavation in limited length, should be given to the Platform section in order to ensure its stability. Results of 7 analyses (see Table 4.3.6) indicates that the selected strut section and/or spacing need to be modified in the lower level strut as the predicted loads exceed the allowable force. The largest difference in these sections is the magnitude of displacements. Figure 4.3.67 shows the maximum wall deflections, surface settlements, and the horizontal displacements as a function of excavation width\textsuperscript{123}. Similar to findings from the parametric study on excavation widths (Section 3.2), displacements increase as the excavation becomes wider and the slope of displacement vs. B/2 also increases with excavation depth.

\textsuperscript{123} Note that the top portion of the Mezzanine Section has an excavation width of 33.5-m (B/2 = 55-ft). At Steps 4 and 5, the excavation width is reduced to 18.3-m (B/2 = 30-ft).
4.4 EVALUATION OF THE PREDICTIVE CAPABILITIES AND LIMITATIONS OF THE NUMERICAL ANALYSES

The performance of three excavations in Boston is analyzed in this chapter: South Cove, South Boston, and the MBTA Transitway Project. All three sites involved excavations of more than 10-m in a BBC-dominant profile. The South Cove and the South Boston sites are excavations which have been completed and the predictive capabilities and the limitations of the numerical analyses are assessed based on comparison with the measured field data. Numerical analyses performed for the Transitway site are a part of the design study to predict the behavior of the proposed braced excavations and to evaluate the proposed support structures.

Detailed analyses of the South Cove and the South Boston sites are presented in Sections 4.1 and 4.2. In general, good predictions of the behavior of diaphragm walls are achieved with the current analyses as shown by the performance of Section A-A at the South Cove project and the north diaphragm wall at the South Boston case. Predictions of surface settlements are also good for braced excavations supported by diaphragm wall. Unfortunately, the same level of performance has not been achieved in the case of sheetpile walls or walls supported by tieback anchors. At the North wall in South Boston, where significant disturbance in the retained soil due to tieback installation was likely, the predicted surface settlements greatly underestimate the field measurements. In both the South Cove and the South Boston cases, the numerical analyses do not capture the large deflections and settlements observed in the sheetpile wall. Possible causes for this limitation in the current numerical analysis include the following:
1. The analyses assume no installation effects for the sheetpile wall. However, the process of driving sheetpile wall induce additional pore pressure within the surrounding clay stratum and is likely to yield a reduction in the clay strength, and consequently yield larger deformations.

2. The analyses assume no tieback installation effects. Seepage of wash fluid and air was observed at the South Boston during the installation of tiebacks installed at the North wall. This installation effect is likely to be the cause for the large settlements observed behind the North wall.

3. The sheetpile wall is more sensitive to the properties of the surrounding soil (cf. Figure 3.4.20). The differences in the observed predicted wall deflections generally occur within soil layers where large uncertainties in soil properties exist. At this time, simple soil models are used for the overlying non-BBC materials and the recommended range for the MIT-E3 model is for BBC with OCR < 4.

4. The numerical analyses assume that the sheetpile wall remains in plane strain. However, movements along the excavation alignment as well as twisting of the sheetpile are possible and would cause the sheetpile to deviate significantly from the plane strain behaviors due to reductions in the effective bending stiffness.

The above possible causes only impact the more flexible sheetpile walls or excavations using tiebacks and are unlikely to affect the behavior of the more rigid diaphragm walls. In addition to the limitation of capturing the behavior of the sheetpile walls, the current analysis also does not account for the tieback installation effects. As shown in the South Boston case, installation of tiebacks can cause significant amount of additional settlement, especially near the excavation.
The Transitway Project involve braced excavation supported mostly by concrete diaphragm wall. Therefore, the limitations mentioned above do not alter the evaluations of the proposed design. Though the predictions from the Original and Revised stress history profiles are likely to be bounds of the actual excavation performance, new input parameters for the MIT-E3 soil model should be selected to match the behavior of natural Boston Blue Clay. This should improve the predictive capability of the numerical analysis, especially for excavations greater than 10-m.
<table>
<thead>
<tr>
<th>Strut Level</th>
<th>Depth m, [ft]</th>
<th>Stiffness MN/m², [kips/ft²]</th>
<th>Preload kN/m, [kips/ft]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Section A-A</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>12.2, [40]</td>
<td>171, [3571]</td>
<td>374, [26]</td>
</tr>
<tr>
<td><strong>Section B-B</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>6.7, [22]</td>
<td>90, [1881]</td>
<td>no preload</td>
</tr>
<tr>
<td>D</td>
<td>11.0, [36]</td>
<td>104, [2173]</td>
<td>no preload</td>
</tr>
</tbody>
</table>

The struts located at Section A-A has a horizontal spacing of 3.6-m [12-ft]
The struts located at Section B-B has a horizontal spacing of 4.5-m [14.8-ft]

Table 4.1.1 - Elastic Properties of Cross-Lot Bracing at South Cove
<table>
<thead>
<tr>
<th>Tieback</th>
<th>Anchor Head Depth, m [Elev., ft]</th>
<th>Tieback Inclination Angle, °</th>
<th>Free Length, m [ft]</th>
<th>Fixed Length, m [ft]</th>
<th>Number of strands</th>
<th>Lateral spacing, m [ft]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>North Side</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tier 1-N</td>
<td>1.2-m [El. 106']</td>
<td>45°</td>
<td>50.3-51.5 [165-169']</td>
<td>9.2-m [30']</td>
<td>9 - 10</td>
<td>3.2-m [10.5']</td>
</tr>
<tr>
<td>Tier 2-N</td>
<td>4.9-m [El. 94']</td>
<td>45°</td>
<td>45.4-46.3 [149-152']</td>
<td>9.2-m [30']</td>
<td>10 - 11</td>
<td>2.0-m [6.5']</td>
</tr>
<tr>
<td>Tier 3-N</td>
<td>8.5-m [El. 82']</td>
<td>45°</td>
<td>38.1-40.2 [125-132']</td>
<td>9.8-m [32']</td>
<td>11 - 12</td>
<td>2.0-m [6.5']</td>
</tr>
<tr>
<td><strong>South Side</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tier 1-S</td>
<td>2.4-m [El. 102']</td>
<td>22°</td>
<td>25.6-26.2 [84-86']</td>
<td>12.2-m [40']</td>
<td>5 - 6</td>
<td>4.4-m [14.5']</td>
</tr>
<tr>
<td>Tier 2-S</td>
<td>6.1-m [El. 90']</td>
<td>22°</td>
<td>16.2-16.8 [53-55']</td>
<td>12.2-m [40']</td>
<td>6</td>
<td>1.8-m [6.0']</td>
</tr>
<tr>
<td>Tier 3-S</td>
<td>9.8-m [El. 78']</td>
<td>20°</td>
<td>7.0-7.6 [23-25']</td>
<td>12.2-m [40']</td>
<td>6</td>
<td>1.8-m [6.0']</td>
</tr>
</tbody>
</table>

Table 4.2.1 - Summary of Tieback Dimensions at ISS-4 [Whelan, p.141, 1995]

<table>
<thead>
<tr>
<th>Tieback</th>
<th>Lock-off Load per Unit Length, kN/m [kips/ft]</th>
<th>Effective Young's Modulus, E, kN/m², [ksi]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>North Side</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tier 1-N</td>
<td>400.0 [27.4]</td>
<td>1.53E8 [22,200]</td>
</tr>
<tr>
<td>Tier 2-N</td>
<td>751.8 [51.5]</td>
<td>1.45E8 [21,100]</td>
</tr>
<tr>
<td>Tier 3-N</td>
<td>944.5 [64.7]</td>
<td>1.38E8 [20,200]</td>
</tr>
<tr>
<td><strong>South Side</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tier 1-S</td>
<td>198.5 [13.6]</td>
<td>1.41E8 [20,500]</td>
</tr>
<tr>
<td>Tier 2-S</td>
<td>443.8 [30.4]</td>
<td>1.17E8 [16,600]</td>
</tr>
<tr>
<td>Tier 3-S</td>
<td>436.5 [29.9]</td>
<td>9.79E7 [14,200]</td>
</tr>
</tbody>
</table>

Table 4.2.2 - Summary of Effective Young's Moduli and Lock-off Loads of Tiebacks at ISS-4 [Whelan, p.141, 1995]
<table>
<thead>
<tr>
<th>Soil Stratum</th>
<th>Property</th>
<th>Value(s) or Range</th>
<th>Source / Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Misc. Fill</td>
<td>(above WT)</td>
<td>17.3</td>
<td>MIT Assumption</td>
</tr>
<tr>
<td>0.0 ~ 2.7-m</td>
<td>$\gamma_t$, kN/m$^3$ [pcf]</td>
<td>18.9</td>
<td>Correlations to grain size and SPT-N values</td>
</tr>
<tr>
<td>[El. 110' ~</td>
<td>(Saturated)</td>
<td>[110]</td>
<td></td>
</tr>
<tr>
<td>El. 101']</td>
<td>$\gamma_t$, kN/m$^3$ [pcf]</td>
<td>[120]</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\phi$</td>
<td>$32^\circ$</td>
<td>Correlations to grain size and SPT-N values</td>
</tr>
<tr>
<td></td>
<td>$K_a$</td>
<td>0.31</td>
<td>Standard Rankine &amp; Jaky equations using $\phi$</td>
</tr>
<tr>
<td></td>
<td>$K_p$</td>
<td>3.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$K_o$</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Permeability, k, m/day</td>
<td>8.6~8.6E-3</td>
<td>10 in-situ pumping tests</td>
</tr>
<tr>
<td>[ft/sec]</td>
<td>[3.3E-4~3.3E-7]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cohesive Fill</td>
<td>$\gamma_t$, kN/m$^3$ [pcf]</td>
<td>17.3</td>
<td>MHD Geo. Cslt. recommendation, based on</td>
</tr>
<tr>
<td>2.7 ~ 7.2-m</td>
<td></td>
<td>[110]</td>
<td>average from lab tests</td>
</tr>
<tr>
<td>[El. 101' ~ El.</td>
<td>Undrained Strength, $C_u$</td>
<td>0.2$\sigma'_p$ ~ 0.3</td>
<td>Based on MIT re-evaluation of test data</td>
</tr>
<tr>
<td>86.5']</td>
<td></td>
<td>$\sigma'_p$ [0.34$\sigma'_v$0 ~ 0.51$\sigma'_v$0)</td>
<td>from MHD Geo. Cslt. Approx. &quot;lower bound&quot; to</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>UU, torvane, and lab vane data.</td>
</tr>
<tr>
<td></td>
<td>OCR</td>
<td>1.7</td>
<td>MIT analysis of MHD Geo. Cslt. oedometer</td>
</tr>
<tr>
<td></td>
<td>CR</td>
<td>0.17</td>
<td>Oedometer data</td>
</tr>
<tr>
<td></td>
<td>RR</td>
<td>0.025</td>
<td>Oedometer data</td>
</tr>
<tr>
<td></td>
<td>$e_0$</td>
<td>1.22</td>
<td>based on $\omega_N = 44%$</td>
</tr>
<tr>
<td></td>
<td>Permeability, k, m/day</td>
<td>2.2E-4~3.0E-3</td>
<td>Oedometer and in-situ tests</td>
</tr>
<tr>
<td>[ft/sec]</td>
<td>[1.2E-7~8.1E-9]</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4.2.3a - Recommended Soil Engineering Properties for ISS-4 [after Whelan, 1995, Tables 5.1 & 5.2]
<table>
<thead>
<tr>
<th>Soil Stratum</th>
<th>Property</th>
<th>Value(s) or Range</th>
<th>Source / Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Organic Deposits</strong></td>
<td>$\gamma_t$, kN/m$^3$ [pcf]</td>
<td>17.0 [108]</td>
<td>MIT recommended value for ISS-4</td>
</tr>
<tr>
<td>7.2 ~ 11.7-m</td>
<td>Undrained Strength, (Top of dep.)</td>
<td></td>
<td>MIT SHANSEP analysis using $S=0.25$ and $m=0.8$, DSS mode of shearing (assuming $\sigma'_p = 3.5$ksf)</td>
</tr>
<tr>
<td></td>
<td>$C_u$, kN/m$^2$ [psf] (Bottom)</td>
<td>34.9 [730]</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$C_u$, kN/m$^2$ [psf] (Bottom)</td>
<td>37.8 [790]</td>
<td></td>
</tr>
<tr>
<td></td>
<td>OCR</td>
<td>1.1 to 2.7</td>
<td>MIT re-evaluation of data</td>
</tr>
<tr>
<td></td>
<td>CR</td>
<td>0.24</td>
<td>Based on mid-range values</td>
</tr>
<tr>
<td></td>
<td>RR</td>
<td>0.02</td>
<td>Based on mid-range values</td>
</tr>
<tr>
<td></td>
<td>$K_0$</td>
<td>0.55(OCR)$^{0.45}$</td>
<td>MIT estimate</td>
</tr>
<tr>
<td></td>
<td>$e_0$</td>
<td>1.46</td>
<td>based on $\omega_N = 55%$</td>
</tr>
<tr>
<td></td>
<td>Permeability, $k$, m/day [ft/sec]</td>
<td>3.5E-5 [1.3E-9]</td>
<td>MIT estimate</td>
</tr>
<tr>
<td><strong>Boston Blue Clay</strong></td>
<td>$\gamma_t$, kN/m$^3$ [pcf]</td>
<td>18.2±0.5 [116±3]</td>
<td>Average from all MHD Geo. Cslt. lab tests</td>
</tr>
<tr>
<td>11.7 ~ 31.4-m</td>
<td>CR</td>
<td>0.17±0.005</td>
<td>Selected by MIT</td>
</tr>
<tr>
<td></td>
<td>(Crust &gt;El.40)</td>
<td>0.23±0.02</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(Lower &lt;El.40)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>[El. 71.5' ~</td>
<td>RR</td>
<td>0.025±0.010</td>
<td>Selected by MIT</td>
</tr>
<tr>
<td>El. 7']</td>
<td>OCR</td>
<td>see Fig. 4.2.5</td>
<td>MIT re-evaluation of consolidation lab test data</td>
</tr>
<tr>
<td></td>
<td>Permeability, $k$, m/day [ft/sec]</td>
<td>(1.1 ± 0.6) E-4 [3.9 ± 2.0]E-9</td>
<td>Based on 22 CRS tests</td>
</tr>
</tbody>
</table>

Table 4.2.3b - Recommended Soil Engineering Properties for ISS-4 [after Whelan, 1995, Tables 5.3 & 5.4]
<table>
<thead>
<tr>
<th>SOIL MOVEMENT INSTRUMENTS</th>
<th>GROUNDWATER INSTRUMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>INC-102</td>
<td>OW-002</td>
</tr>
<tr>
<td>DMP4-120</td>
<td>VWPZ-108</td>
</tr>
<tr>
<td>DMP2-006</td>
<td>VWPZ-067/068</td>
</tr>
<tr>
<td>DMP2-107</td>
<td>VWPZ-107</td>
</tr>
<tr>
<td>DMP2-105</td>
<td></td>
</tr>
<tr>
<td>DMP2-104</td>
<td></td>
</tr>
<tr>
<td>DMP2-070</td>
<td></td>
</tr>
<tr>
<td>North of Diaphragm Wall</td>
<td></td>
</tr>
<tr>
<td>INC-101</td>
<td>MPHG-110</td>
</tr>
<tr>
<td>IPE-113</td>
<td>VWPZ-135/136</td>
</tr>
<tr>
<td>DMP4-118</td>
<td>MPHG109</td>
</tr>
<tr>
<td>DMP4-117</td>
<td>VWPZ-133/134</td>
</tr>
<tr>
<td>DMP4-116</td>
<td>MPHG501</td>
</tr>
<tr>
<td></td>
<td>VWPZ-131/132</td>
</tr>
<tr>
<td></td>
<td>MPHG107</td>
</tr>
<tr>
<td></td>
<td>VWPZ-053</td>
</tr>
<tr>
<td>South of Sheetpile Wall</td>
<td></td>
</tr>
<tr>
<td>INC-101</td>
<td>IPE-113</td>
</tr>
<tr>
<td>IPE-113</td>
<td>VWPZ-106</td>
</tr>
<tr>
<td></td>
<td>VWPZ-106</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

INC  Inclinometers
DMP  Deflection Monitoring Points
OW   Observation Wells
VWPZ  Vibrating Wire Piezometers
IPE  Inclinometer/Probe Extensometers
MPHG Multi-Point Heave Gages
OSPZ Open Standpipe Piezometers

Table 4.2.4 - Geotechnical Field Instruments at ISS-4, Boston
## Table 4.2.5 - Soil Model and Input Material Properties for Soils in the Upper Aquifer

<table>
<thead>
<tr>
<th>Soil Layers</th>
<th>Misc. Fill</th>
<th>Cohesive Fill</th>
<th>Organic Deposits</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Stratum Location</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth, m</td>
<td>0.0 - 1.2</td>
<td>1.2 - 2.7</td>
<td>7.2 - 10.4</td>
</tr>
<tr>
<td>Elevation, ft</td>
<td>[El. 110-106']</td>
<td>[El. 106-101']</td>
<td>[El. 86.5-76']</td>
</tr>
<tr>
<td><strong>Unit Weight, ( \gamma_v )</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>kN/m³; [pcf]</td>
<td>17.3</td>
<td>18.9</td>
<td>17.0</td>
</tr>
<tr>
<td>[110]</td>
<td>[120]</td>
<td>[108]</td>
<td></td>
</tr>
<tr>
<td><strong>K₀</strong></td>
<td>0.5</td>
<td>0.65</td>
<td>0.733</td>
</tr>
<tr>
<td></td>
<td>0.3</td>
<td>0.3</td>
<td>0.688</td>
</tr>
<tr>
<td><strong>ν'</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Norm. Shear Modulus, ( G/\sigma_v )</strong></td>
<td>35</td>
<td>33</td>
<td>44 to 42</td>
</tr>
<tr>
<td><strong>Norm. Shear Modulus, ( G/S_u )</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>-----</td>
<td>96.7</td>
<td>105.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>113.4</td>
</tr>
<tr>
<td><strong>Norm. Undrain. Strength, ( S_{uPS}/\sigma_v )</strong></td>
<td></td>
<td>0.34</td>
<td>0.42</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.37</td>
</tr>
<tr>
<td><strong>Drained Friction, ( \phi_{PS} (\degree) )</strong></td>
<td>32°</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Permeability, ( k ), m/day; [ft/sec]</strong></td>
<td></td>
<td>4.345</td>
<td>4.32E-3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>[1.7E-4]</td>
<td>[1.7E-7]</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>[1.7E-8]</td>
</tr>
</tbody>
</table>

Note: (1) Total unit weight for \( \gamma_v = 18.2 \; \text{kN/m}^3 \) [116pcf]  
(2) Average Permeability = \( k = 4.32E-5 \; \text{m/day} \) [1.64E-9 ft/sec]

## Table 4.2.6 - Input Stress History and In-Situ Stress State for BBC layer using MIT-E3 Soil Model

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Elevation (ft)</th>
<th>OCR</th>
<th>( K_0 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.7 to 11.9</td>
<td>71.5 to 71.0</td>
<td>5.8</td>
<td>1.26</td>
</tr>
<tr>
<td>11.9 to 12.5</td>
<td>71.0 to 69.0</td>
<td>5.2</td>
<td>1.17</td>
</tr>
<tr>
<td>12.5 to 14.6</td>
<td>69.0 to 62.0</td>
<td>4.5</td>
<td>1.07</td>
</tr>
<tr>
<td>14.6 to 16.8</td>
<td>62.0 to 55.0</td>
<td>3.5</td>
<td>0.91</td>
</tr>
<tr>
<td>16.8 to 19.2</td>
<td>55.0 to 47.0</td>
<td>2.8</td>
<td>0.81</td>
</tr>
<tr>
<td>19.2 to 21.6</td>
<td>47.0 to 39.0</td>
<td>2.4</td>
<td>0.74</td>
</tr>
<tr>
<td>21.6 to 24.1</td>
<td>39.0 to 31.0</td>
<td>2.0</td>
<td>0.69</td>
</tr>
<tr>
<td>24.1 to 26.5</td>
<td>31.0 to 23.0</td>
<td>1.8</td>
<td>0.66</td>
</tr>
<tr>
<td>26.5 to 29.0</td>
<td>23.0 to 15.0</td>
<td>1.7</td>
<td>0.64</td>
</tr>
<tr>
<td>29.0 to 31.4</td>
<td>15.0 to 7.0</td>
<td>1.5</td>
<td>0.62</td>
</tr>
</tbody>
</table>

Note: (1) Total unit weight for BBC = \( \gamma_v = 18.2 \; \text{kN/m}^3 \) [116pcf]  
(2) Average Permeability = \( k = 4.32E-5 \; \text{m/day} \) [1.64E-9 ft/sec]
<table>
<thead>
<tr>
<th>STRUCTURAL ELEMENT</th>
<th>INPUT PARAMETERS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. North Diaphragm Wall</td>
<td>Solid Elastic Elements with:</td>
</tr>
<tr>
<td>- 0.9-m [3.0-ft] thick reinforced concrete</td>
<td>$\nu = 0.15$</td>
</tr>
<tr>
<td></td>
<td>$E = 2.26E7 \text{ kN/m}^2 = 4.71E5 \text{ kips/ft}^2$</td>
</tr>
<tr>
<td>2. South Sheetpile Wall</td>
<td>Solid Elastic Elements with:</td>
</tr>
<tr>
<td>- Arbed AZ-18 Sheet Piling</td>
<td>Equivalent thickness#</td>
</tr>
<tr>
<td>(see Figure 4.2.4)</td>
<td>$t_{fe} = 0.52 \text{-m} = 1.7 \text{-ft}$</td>
</tr>
<tr>
<td></td>
<td>Equivalent Modulus#</td>
</tr>
<tr>
<td></td>
<td>$E_{fe} = 5.73E6 \text{ kN/m}^2 = 1.2E5 \text{ kips/ft}^2$</td>
</tr>
<tr>
<td></td>
<td>$\nu = 0.10$</td>
</tr>
<tr>
<td>3. North Wall Tiebacks</td>
<td>One Elastic Spring Element with:</td>
</tr>
<tr>
<td>a. Tier 1-N</td>
<td>$Spring \ Stiffness \ for \ Tier \ 1-N$:</td>
</tr>
<tr>
<td></td>
<td>$k_{1-N} = 1.25E3 \text{ kN/m/m} = 26.1 \text{ kips/ft/ft}$</td>
</tr>
<tr>
<td>b. Tier 2-N</td>
<td>$Spring \ Stiffness \ for \ Tier \ 2-N$:</td>
</tr>
<tr>
<td></td>
<td>$k_{2-N} = 2.36E3 \text{ kN/m/m} = 49.3 \text{ kips/ft/ft}$</td>
</tr>
<tr>
<td>c. Tier 3-N</td>
<td>$Spring \ Stiffness \ for \ Tier \ 3-N$:</td>
</tr>
<tr>
<td></td>
<td>$k_{3-N} = 2.86E3 \text{ kN/m/m} = 59.6 \text{ kips/ft/ft}$</td>
</tr>
<tr>
<td>4. South Wall Tiebacks</td>
<td>Four Elastic Spring Elements* in series:</td>
</tr>
<tr>
<td>a. Tier 1-S</td>
<td>1 representing free length</td>
</tr>
<tr>
<td>(free length)</td>
<td>3 representing fixed length</td>
</tr>
<tr>
<td>(fixed length)</td>
<td>$Spring \ Stiffness \ for \ Tier \ 1-S$:</td>
</tr>
<tr>
<td></td>
<td>$k_{1-S}^{\text{(free)}} = 1.2E3 \text{ kN/m/m} = 25.9 \text{ kips/ft/ft}$</td>
</tr>
<tr>
<td></td>
<td>$k_{1-S}^{\text{(fixed)}} = 7.9E3 \text{ kN/m/m} = 165 \text{ kips/ft/ft}$</td>
</tr>
<tr>
<td>b. Tier 2-S</td>
<td>$Spring \ Stiffness \ for \ Tier \ 2-S$:</td>
</tr>
<tr>
<td>(free length)</td>
<td>$k_{2-S}^{\text{(free)}} = 4.2E3 \text{ kN/m/m} = 87.0 \text{ kips/ft/ft}$</td>
</tr>
<tr>
<td>(fixed length)</td>
<td>$k_{2-S}^{\text{(fixed)}} = 1.7E4 \text{ kN/m/m} = 352 \text{ kips/ft/ft}$</td>
</tr>
<tr>
<td>c. Tier 3-S</td>
<td>$Spring \ Stiffness \ for \ Tier \ 3-S$:</td>
</tr>
<tr>
<td>(free length)</td>
<td>$k_{3-S}^{\text{(free)}} = 8.0E3 \text{ kN/m/m} = 167.4 \text{ kips/ft/ft}$</td>
</tr>
<tr>
<td>(fixed length)</td>
<td>$k_{3-S}^{\text{(fixed)}} = 1.4E4 \text{ kN/m/m} = 301 \text{ kips/ft/ft}$</td>
</tr>
</tbody>
</table>

NOTE: * see Figure 2.4.10

# equivalent thickness and equivalent modulus for a solid element with the same bending and axial stiffnesses as the AZ-18 section [after Potts and Day, 1993]

Table 4.2.7 - Properties and Input Parameters for Structural Elements of Lateral Earth Support System
<table>
<thead>
<tr>
<th>Construction Events</th>
<th>Const. Days [CD]</th>
<th>Whelan's Step Definition</th>
<th>FE Analysis Step Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dec. 7 - 9, 1992</td>
<td>Drive SPW</td>
<td></td>
<td>Step 1</td>
</tr>
<tr>
<td>Mar. 22, 1993</td>
<td>Excavate 1-S</td>
<td>0</td>
<td>Step 1S, Mar. 22, 1993</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>to Step 1S-b, May 7, 1993</td>
</tr>
<tr>
<td>Mar. 24- Apr.13</td>
<td>Install DW</td>
<td>2 to 22</td>
<td>Step 1N, May 7, 1993</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>to Step 1N-b, June 5, 1993</td>
</tr>
<tr>
<td>Apr. 13, 1993</td>
<td>Install 1-S TB</td>
<td>22</td>
<td>Step 2</td>
</tr>
<tr>
<td>May 4 - 5, 1994</td>
<td>Excavate 1-N</td>
<td>43 - 44</td>
<td>Step 2b, July 1, 1993</td>
</tr>
<tr>
<td>May 17 - 21</td>
<td>Excavate 2-C</td>
<td>56 - 60</td>
<td>Step 3</td>
</tr>
<tr>
<td>May 18 - 24</td>
<td>Install 1-N TB</td>
<td>57 - 63</td>
<td>Step 3, July 1, 1993</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>to Step 3b, Aug. 9, 1993</td>
</tr>
<tr>
<td>May 28 - June 1</td>
<td>Lock-off 1-N TB</td>
<td>67 - 71</td>
<td>Step 4</td>
</tr>
<tr>
<td>June 1 - 3, 1993</td>
<td>Excavate 2-N</td>
<td>71 - 73</td>
<td>Step 4, Aug. 9, 1993</td>
</tr>
<tr>
<td>June 5, 1993</td>
<td>Lock-off 1-S TB</td>
<td>75</td>
<td>Step 5</td>
</tr>
<tr>
<td>June 7 - 14</td>
<td>Install 2-N TB</td>
<td>77 - 84</td>
<td>Step 5, Aug. 20, 1993</td>
</tr>
<tr>
<td>June 15, 1993</td>
<td>Excavate 3-S</td>
<td>85</td>
<td>Step 6</td>
</tr>
<tr>
<td>June 25, 1993</td>
<td>Excavate 3-C</td>
<td>95</td>
<td>Step 6, Sept 15, 1993</td>
</tr>
<tr>
<td>June 30 - July 1</td>
<td>Install 2-S TB</td>
<td>100 - 101</td>
<td>Step 6, Sept 15, 1993</td>
</tr>
<tr>
<td>July 1 - 2, 1993</td>
<td>Lock-off 2-N TB</td>
<td>101 - 102</td>
<td>Step 7</td>
</tr>
<tr>
<td>July 12 - 13</td>
<td>Excavate 4-N</td>
<td>112 - 113</td>
<td>Step 7, Mar. 2, 1994</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>to Step 7b, Mar. 2, 1994</td>
</tr>
<tr>
<td>July 30 - 31</td>
<td>Lock-off 2-S TB</td>
<td>130 - 131</td>
<td></td>
</tr>
<tr>
<td>July 27 - Aug. 5</td>
<td>Install 3-N TB</td>
<td>151</td>
<td></td>
</tr>
<tr>
<td>Aug. 11 - 16</td>
<td>Excavate 5-S</td>
<td>142 - 147</td>
<td></td>
</tr>
<tr>
<td>Aug. 16 - 17</td>
<td>Install 3-S TB</td>
<td>147 - 148</td>
<td></td>
</tr>
<tr>
<td>Aug. 20, 1993</td>
<td>Lock-off 3-N TB</td>
<td>151</td>
<td></td>
</tr>
<tr>
<td>Sept. 10 - 30</td>
<td>Jet Grout TD</td>
<td>172 - 192</td>
<td></td>
</tr>
<tr>
<td>Sept. 17 - 20</td>
<td>Lock-off 3-S TB</td>
<td>179 - 182</td>
<td></td>
</tr>
<tr>
<td>Sept. 28 - 30</td>
<td>Excavate 6-C</td>
<td>190 - 192</td>
<td></td>
</tr>
<tr>
<td>Oct. 13 - 14, 1993</td>
<td>Grade Haul Road</td>
<td>205 - 206</td>
<td></td>
</tr>
<tr>
<td>Dec. 1993</td>
<td>Drive TD casing</td>
<td>&lt; 254</td>
<td></td>
</tr>
<tr>
<td>Mar. 2 - 4, 1994</td>
<td>Excavate tier 7</td>
<td>345 - 347</td>
<td>Step 7</td>
</tr>
<tr>
<td>Mar. 17, 1994</td>
<td>Pour S. Invert</td>
<td>360</td>
<td>Step 8</td>
</tr>
</tbody>
</table>

**NOTE:**
- SPW: South Sheetpile Wall
- DW: North Sheetpile Wall
- TB: Tiebacks
- 2-N: Tier 2, North Side
- 2-C: Tier 2, Center
- 3-S: Tier 3, South Side
- TD: Tiedowns

Shaded entry indicates that Construction Events NOT incorporated in the FE model.

**Table 4.2.8 - Summary of Actual Construction Sequence and Numerical Model Construction Sequence Definitions**
<table>
<thead>
<tr>
<th>Figure Content</th>
<th>Measured Data [Whelan, 1995]</th>
<th>Base Case Results</th>
<th>Revised Case Results</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Wall Deflections and Surface Settlement:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Step 1</td>
<td>[Figure 7B.1(b)]</td>
<td>Figures 4.2.13 and 4.2.14</td>
<td>Figures 4.2.38</td>
</tr>
<tr>
<td>Step 2</td>
<td>[Figure 7B.2(b)]</td>
<td>Figure 4.2.15</td>
<td>Figure 4.2.39</td>
</tr>
<tr>
<td>Step 3</td>
<td>[Figure 7B.3(b)]</td>
<td>Figure 4.2.16</td>
<td>Figure 4.2.40</td>
</tr>
<tr>
<td>Step 4</td>
<td>[Figure 7B.4(b)]</td>
<td>Figure 4.2.17</td>
<td>Figure 4.2.41</td>
</tr>
<tr>
<td>Step 5</td>
<td>[Figure 7B.5(b)]</td>
<td>Figure 4.2.18</td>
<td>Figure 4.2.42</td>
</tr>
<tr>
<td>Step 6</td>
<td>[Figure 7B.6(b)]</td>
<td>Figure 4.2.19</td>
<td>Figure 4.2.43</td>
</tr>
<tr>
<td>Step 7</td>
<td>[Figure 7B.7(b)]</td>
<td>Figure 4.2.20</td>
<td>---</td>
</tr>
<tr>
<td><strong>Pore Pressure vs. Time</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>at Deep Wells</td>
<td>[Figure 7A.7]</td>
<td>Figure 4.2.12</td>
<td>Figure 4.2.12</td>
</tr>
<tr>
<td>North OWs</td>
<td>[Figure 7A.8]</td>
<td>Figure 4.2.21</td>
<td>Figure 4.2.45</td>
</tr>
<tr>
<td>North VWPZ</td>
<td>[Figure 7A.9]</td>
<td>Figure 4.2.22</td>
<td>Figure 4.2.46</td>
</tr>
<tr>
<td>South OSPZ</td>
<td>[Figure 7A.10]</td>
<td>Figure 4.2.23</td>
<td>Figure 4.2.47</td>
</tr>
<tr>
<td>inside Excavation</td>
<td>[Figures 7A.15, 7A.16, &amp; 7A.17]</td>
<td>Figures 4.2.24, 4.2.25, &amp; 4.2.26</td>
<td>Figures 4.2.48, 4.2.49, &amp; 4.2.50</td>
</tr>
<tr>
<td><strong>Heave within Excavation vs. Time:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MPHG-110</td>
<td>[Figure 7A.11]</td>
<td>Figure 4.2.27</td>
<td>Figure 4.2.51</td>
</tr>
<tr>
<td>MPHG-109</td>
<td>[Figure 7A.12]</td>
<td>Figure 4.2.28</td>
<td>Figure 4.2.52</td>
</tr>
<tr>
<td>MPHG-501</td>
<td>[Figure 7A.13]</td>
<td>Figure 4.2.29</td>
<td>Figure 4.2.53</td>
</tr>
<tr>
<td>MPHG-107</td>
<td>[Figure 7A.14]</td>
<td>Figure 4.2.30</td>
<td>Figure 4.2.54</td>
</tr>
<tr>
<td><strong>Wall Deflection vs. Time:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>INC-102</td>
<td>[Figure 7A.1]</td>
<td>Figure 4.2.31</td>
<td>Figure 4.2.55</td>
</tr>
<tr>
<td>INC-101</td>
<td>[Figure 7A.2]</td>
<td>Figure 4.2.32</td>
<td>Figure 4.2.56</td>
</tr>
<tr>
<td>IPE-113</td>
<td>[Figure 7A.3]</td>
<td>Figure 4.2.33</td>
<td>Figure 4.2.57</td>
</tr>
<tr>
<td><strong>Surface Settlement vs. Time:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>North Side</td>
<td>[Figure 7A.4]</td>
<td>Figure 4.2.34</td>
<td>Figure 4.2.58</td>
</tr>
<tr>
<td>South Side</td>
<td>[Figure 7A.5]</td>
<td>Figure 4.2.35</td>
<td>Figure 4.2.59</td>
</tr>
<tr>
<td>IPE-113</td>
<td>[Figure 7A.3]</td>
<td>Figure 4.2.36</td>
<td>Figure 4.2.60</td>
</tr>
</tbody>
</table>

NOTE: Figure numbers for measured data refer to Figures in Whelan, 1995

Table 4.2.9 - Listing of Figures Showing Results of the Base and Revised Case Numerical Analyses
<table>
<thead>
<tr>
<th>Transitway Section</th>
<th>Excavation Width, B, m [ft]</th>
<th>Final Excavation Depth, H m [ft]</th>
<th>Support Wall Type</th>
<th>Wall Length, L m [ft]</th>
<th>Strut Spacing, h m [ft]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Platform</td>
<td>33.5-m [110-ft]</td>
<td>17.7-m [58-ft]</td>
<td>0.9-m DW</td>
<td>26.8/23.8-m [88/78-ft]</td>
<td>3.0/4.0-m [10 / 13-ft]</td>
</tr>
<tr>
<td>Mezzanine</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top</td>
<td>33.5-m [110-ft]</td>
<td>10.4-m [35-ft]</td>
<td>0.9-m DW</td>
<td>17.7-m [58-ft]</td>
<td>4.0-m [13-ft]</td>
</tr>
<tr>
<td>Bottom</td>
<td>18.3-m [60-ft]</td>
<td>17.7-m [58-ft]</td>
<td>PZ38 SPW</td>
<td>20.7-m [68-ft]</td>
<td></td>
</tr>
<tr>
<td>Transition</td>
<td>16.5-m [54-ft]</td>
<td>17.7-m [58-ft]</td>
<td>0.9-m DW</td>
<td>26.8-m [88-ft]</td>
<td>4.0-m [13-ft]</td>
</tr>
<tr>
<td>West Tunnel</td>
<td>9.8-m [32-ft]</td>
<td>18.6-m [61-ft]</td>
<td>0.9-m DW</td>
<td>26.2-m [86-ft]</td>
<td>4.0-m [13-ft]</td>
</tr>
</tbody>
</table>

Table 4.3.1 - Summary of Excavation Geometries for the Four Typical Cross-Sections at the Transitway Project
<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Miscellaneous Fill</th>
<th>Cohesive Fill</th>
<th>Silty Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strata Location</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth, m</td>
<td>0 - 1.8</td>
<td>1.8 - 8.5</td>
<td>8.5 - 12.2</td>
</tr>
<tr>
<td>Elevation, [ft]</td>
<td>[El.112'-106']</td>
<td>[El.106'-84']</td>
<td>[El. 84'-72']</td>
</tr>
<tr>
<td>Unit Weight, ( \gamma_t ) kN/m³</td>
<td>18.9 [120]</td>
<td>18.1 [115]</td>
<td>18.9 [120]</td>
</tr>
<tr>
<td></td>
<td>[pcf]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>K₀</td>
<td>0.5</td>
<td>0.75</td>
<td>0.5</td>
</tr>
<tr>
<td>( v' )</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Normalized Shear Modulus, ( G/\sigma'_{v0} )</td>
<td>35</td>
<td>20</td>
<td>35</td>
</tr>
<tr>
<td>Undrained Shear Strength, ( S_{uPS} ) kN/m² [ksf]</td>
<td>El.106-99': 7.2 [0.15]</td>
<td>El. 99-91': 12.5 [0.26]</td>
<td>El. 91-84': 17.2 [0.36]</td>
</tr>
<tr>
<td>Drained Friction Angle, ( \phi'_{PS}, \circ )</td>
<td>30</td>
<td>---</td>
<td>35</td>
</tr>
<tr>
<td>Permeability, k m/day</td>
<td>4.3 [1.65E-4]</td>
<td>8.7E-3 [3.3E-7]</td>
<td>4.3 [3.3E-5]</td>
</tr>
</tbody>
</table>

Table 4.3.2 - Properties and Model Parameters of Upper Soil Layers
<table>
<thead>
<tr>
<th>Structural Element</th>
<th>Input Parameters</th>
<th>Design Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Bracing Strut: (Pipe Section)</td>
<td>Equivalent Spring Stiffness: $K_{eq} = \frac{AE}{(B/2)S} = \frac{90960}{(B/2)} kips/ft$</td>
<td>Design Load:</td>
</tr>
<tr>
<td>- E=29,000 ksi steel</td>
<td></td>
<td>$F_{(design)} = 657 kN/m$</td>
</tr>
<tr>
<td>- 30&quot; O.D.</td>
<td></td>
<td>[45 kips/ft]</td>
</tr>
<tr>
<td>- 0.75&quot; thick</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- A = 69 in$^2$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- S = 22-ft (horiz. spacing)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. Diaphragm Wall</td>
<td>$E = 22.6E6$ kN/m$^2$</td>
<td>Allowable Moment:</td>
</tr>
<tr>
<td>- 3-ft thick reinf. concrete</td>
<td>= 4.71E5 kips/ft$^2$</td>
<td>$M_{all}=1.2$ MN-m/m</td>
</tr>
<tr>
<td></td>
<td>$\nu = 0.15$</td>
<td>265 kip-ft/ft</td>
</tr>
<tr>
<td>3. Sheet Pile Wall</td>
<td>Equivalent Thickness*: $T_{eq} = 0.4$-m = 1.4-ft</td>
<td>Allowable Moment:</td>
</tr>
<tr>
<td>(Mezzanine Only)</td>
<td>Equivalent Modulus*: $E_{eq} = 1.1E4$ MN/m$^2$</td>
<td>$M_{all}=0.4$ MN-m/m</td>
</tr>
<tr>
<td>- PZ-38 Section</td>
<td>= 2.24E5 kips/ft$^2$</td>
<td>97 kip-ft/ft</td>
</tr>
<tr>
<td></td>
<td>$\nu = 0.10$</td>
<td></td>
</tr>
</tbody>
</table>

* Selected to match bending and axial stiffness of PZ-38 section [after Potts and Day, 1993]

Table 4.3.3 - Properties and Input Parameters for Structural Elements of Lateral Earth Support System
<table>
<thead>
<tr>
<th>Section</th>
<th>Analysis Name</th>
<th>Strut Spacing (ft)</th>
<th>Wall Toe Elevation (ft)</th>
<th>Drainage Condition</th>
<th>Soil Profile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Platform</td>
<td>P1-O-UD</td>
<td>10</td>
<td>24</td>
<td>Undrained</td>
<td>Original</td>
</tr>
<tr>
<td></td>
<td>P2-O-UD</td>
<td>13</td>
<td>24</td>
<td>Undrained</td>
<td>Original</td>
</tr>
<tr>
<td></td>
<td>P3-O-UD</td>
<td>13</td>
<td>34</td>
<td>Undrained</td>
<td>Original</td>
</tr>
<tr>
<td></td>
<td>P1-O-PD</td>
<td>10</td>
<td>24</td>
<td>Partially D.</td>
<td>Original</td>
</tr>
<tr>
<td></td>
<td>P3-O-PD</td>
<td>13</td>
<td>34</td>
<td>Partially D.</td>
<td>Original</td>
</tr>
<tr>
<td></td>
<td>P3-R-PD</td>
<td>13</td>
<td>34</td>
<td>Partially D.</td>
<td>Revised</td>
</tr>
<tr>
<td>Mezzanine</td>
<td>M1-O-PD</td>
<td>13</td>
<td>DW:54</td>
<td>Partially D.</td>
<td>Original</td>
</tr>
<tr>
<td></td>
<td>M1-R-PD</td>
<td>13</td>
<td>SPW: 44</td>
<td>Partially D.</td>
<td>Revised</td>
</tr>
<tr>
<td>Transition</td>
<td>T1-O-PD</td>
<td>13</td>
<td>24</td>
<td>Partially D.</td>
<td>Original</td>
</tr>
<tr>
<td></td>
<td>T1-R-PD</td>
<td>13</td>
<td>24</td>
<td>Partially D.</td>
<td>Revised</td>
</tr>
<tr>
<td>West Tunnel</td>
<td>W1-O-PD</td>
<td>13</td>
<td>26</td>
<td>Partially D.</td>
<td>Original</td>
</tr>
<tr>
<td></td>
<td>W1-R-PD</td>
<td>13</td>
<td>26</td>
<td>Partially D.</td>
<td>Revised</td>
</tr>
</tbody>
</table>

Table 4.3.4 - Summary of Finite Element Analyses Performed for the MBTA Transitway Excavations

<table>
<thead>
<tr>
<th>Analysis Name</th>
<th>$\delta_{w(\text{max})}$</th>
<th>$\delta_{v(\text{max})}$</th>
<th>$\delta_{h(\text{max})}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>cm</td>
<td>cm</td>
<td>cm</td>
</tr>
<tr>
<td></td>
<td>[in]</td>
<td>[in]</td>
<td>[in]</td>
</tr>
<tr>
<td><strong>Platform Section</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P1-O-UD</td>
<td>6.3 [2.5]</td>
<td>3.4 [1.3]</td>
<td>4.1 [1.6]</td>
</tr>
<tr>
<td>P2-O-UD</td>
<td>6.4 [2.5]</td>
<td>3.5 [1.4]</td>
<td>4.2 [1.6]</td>
</tr>
<tr>
<td>P3-O-UD</td>
<td>6.5 [2.6]</td>
<td>3.5 [1.4]</td>
<td>4.2 [1.7]</td>
</tr>
<tr>
<td>P1-O-PD</td>
<td>4.6 [1.8]</td>
<td>1.7 [0.7]</td>
<td>2.6 [1.0]</td>
</tr>
<tr>
<td>P3-O-PD</td>
<td>4.8 [1.9]</td>
<td>1.9 [0.7]</td>
<td>2.8 [1.1]</td>
</tr>
<tr>
<td>P3-R-PD</td>
<td>8.8 [3.5]</td>
<td>4.0 [1.6]</td>
<td>4.6 [1.8]</td>
</tr>
<tr>
<td><strong>Mezzanine Section</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>M1-O-PD</td>
<td>3.9 [1.5]</td>
<td>1.1 [0.5]</td>
<td>2.1 [0.8]</td>
</tr>
<tr>
<td>M1-R-PD</td>
<td>6.8 [2.7]</td>
<td>3.0 [1.2]</td>
<td>3.8 [1.5]</td>
</tr>
<tr>
<td><strong>Transition Section</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T1-O-PD</td>
<td>3.6 [1.4]</td>
<td>1.0 [0.4]</td>
<td>1.8 [0.7]</td>
</tr>
<tr>
<td>T1-R-PD</td>
<td>6.5 [2.5]</td>
<td>2.5 [1.0]</td>
<td>3.0 [1.2]</td>
</tr>
<tr>
<td><strong>West Tunnel Section</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W1-O-PD</td>
<td>3.1 [1.2]</td>
<td>0.7 [0.3]</td>
<td>1.4 [0.5]</td>
</tr>
<tr>
<td>W1-R-PD</td>
<td>6.0 [2.4]</td>
<td>2.1 [0.8]</td>
<td>2.5 [1.0]</td>
</tr>
</tbody>
</table>

Table 4.3.5 - Summary of Maximum Displacements for the Transitway Project
<table>
<thead>
<tr>
<th>Analysis Name</th>
<th>Moment in Wall $&lt; M_{all}$ Figure</th>
<th>Strut Force $&lt; F_{all}$ Figure</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Platform Section</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P1-O-UD</td>
<td>YES</td>
<td>4.3.17</td>
</tr>
<tr>
<td>P2-O-UD</td>
<td>YES</td>
<td>4.3.20</td>
</tr>
<tr>
<td>P3-O-UD</td>
<td>YES</td>
<td>4.3.26</td>
</tr>
<tr>
<td>P1-O-PD</td>
<td>YES</td>
<td>4.3.32</td>
</tr>
<tr>
<td>P3-O-PD</td>
<td>YES</td>
<td>4.3.38</td>
</tr>
<tr>
<td>P3-R-PD</td>
<td>YES</td>
<td>4.3.44</td>
</tr>
<tr>
<td><strong>Mezzanine Section</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>M1-O-PD</td>
<td>YES</td>
<td>4.3.51</td>
</tr>
<tr>
<td>M1-R-PD</td>
<td>YES</td>
<td>4.3.51</td>
</tr>
<tr>
<td><strong>Transition Section</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T1-O-PD</td>
<td>YES</td>
<td>4.3.56</td>
</tr>
<tr>
<td>T1-R-PD</td>
<td>YES</td>
<td>4.3.56</td>
</tr>
<tr>
<td><strong>West Tunnel Section</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>W1-O-PD</td>
<td>YES</td>
<td>4.3.62</td>
</tr>
<tr>
<td>W1-R-PD</td>
<td>YES</td>
<td>4.3.62</td>
</tr>
</tbody>
</table>

NOTE: $M_{all}$ The allowable moment
for diaphragm wall, $M_{all} = 1.2MN\cdot m/m = 265$ kips-ft/ft
for sheetpile wall, $M_{all} = 0.4Mn\cdot m/m = 97$ kips-ft/ft

$F_{all}$ The allowable strut force = 657 kN/m = 45 kips/ft

Table 4.3.6 - Summary of Structural Loads for the Transitway Project
Figure 4.1.3 - Cross-section at South Cove Section A-A, Station 113+40 (Jaworski, 1973)
Figure 4.1.4 – Cross-section at Section B-B, Station 111+40, South Cove Project (Jaworski, 1973)
Figure 4.1.5 - Soil Profile and In-situ Stress Conditions at South Cove, Section A-A
Figure 4.1.6 - Finite Element Representation of Excavation History for South Cove, Section A-A
Construction Sequence at Section B-B

June 6, 1969 is Construction Day 0 (CD 0)

June 6, 1969
Step 2
2.13m

June 23, 1969
Step 3
1.28m

June 25, 1969
Step 4
3.66m

July 9, 1969
Step 5
4.88m

July 10, 1969
Step 6
6.71m

July 21, 1969
Step 7
10.4m

July 22, 1969
Step 8
11.58m

Aug. 12, 1969
Step 9
11.58m

Aug. 14, 1969
Step 10

Aug. 15, 1969
Step 11
14.6m

Figure 4.1.9 - Finite Element Representation of Excavation History for South Cove, Section B-B
South Cove Section A-A, Sta. 113+40

358 Elements, 1149 Nodes, 2667 D.O.F.

Wall: Solid Element with 8 displacement nodes

Soil: Porous Element with 8 displacement nodes and 4 pore pressure nodes

Strut: Spring element

Pore Pressure Boundary Conditions:

INITIALLY: Hydrostatic with WT at a depth of 5.2m [17ft]. Pore pressure=0 above the WT.

FOR EACH EXCAVATION STEP: Constant head (equivalent to the hydrostatic pore pressures) is imposed on RHS and BOT. At the bottom of the excavation, pore pressure is set to zero.

Figure 4.1.10 – Finite Element Model for South Cove Section A-A
**Step A: Prior to Strut Installation**

**DEFINITION:**
- Node A is a virtual node connected to Node B only
- Node B is a node on the wall —— location of the strut

**CONSTRAINT 1:**
\[ \text{Displacement}_{A} = \text{Displacement}_{B} \]

**Step B: Strut Installation**

Remove Constraint 1

Impose Pre-Load [F/L]

Activate Spring Stiffness [F/L^2]

**Step C: After Strut Installation**

**CONSTRAINT 2:**
- Horizontal Displacement \[ \text{Horizontal Disp}_{A} = 0 \]
- Vert. Disp \[ \text{Vert. Disp}_{A} = \text{Vert. Disp}_{B} \]

---

Figure 4.1.11 - Simulation of the installation and prestress of Cross-Lot Bracing

Page 479
Figure 4.1.12 - Comparison of Predicted and Measured Wall Deflections and Surface Settlements for South Cove Section A-A at Excavation Step 2
Figure 4.1.13 - Comparison of Predicted and Measured Wall Deflections and Surface Settlements for South Cove Section A-A at Excavation Step 6

Page 481
Figure 4.1.14 - Comparison of Predicted and Measured Wall Deflections and Surface Settlements for South Cove Section A-A at Excavation Step 7
Figure 4.1.15 - Comparison of Predicted and Measured Wall Deflections and Surface Settlements for South Cove Section A-A at Excavation Step 8
Figure 4.1.16 - Comparison of Predicted and Measured Pore Pressures for South Cove, Section A-A
South Cove Project
Section A-A

Strut Level B

- Finite Element Analysis
- Partially Drained
- Undrained
- Design Strut Load
- Measured Strut Load

Design Load = 606 kN/m

Strut Level C

Design Load = 435 kN/m

Strut Level D

Design Strut Load at 889 kN/m

Month, 1969

Figure 4.1.17 - Comparison of Predicted and Measured Strut Loads for South Cove, Section A-A
Figure 4.1.19 - Comparison of Predicted and Measured Wall Deflections for South Cove Section B-B at Excavation Step 6
Figure 4.1.20 - Comparison of Predicted and Measured Wall Deflections for South Cove Section B-B at Excavation Step 8
Figure 4.1.21 - Comparison of Predicted and Measured Wall Deflections for South Cove Section B-B at Excavation Step 9
Figure 4.1.22 - Comparison of Predicted and Measured Wall Deflections for South Cove Section B-B at Excavation Step 11
Figure 4.1.23 - Comparison of Predicted and Measured Pore Pressures for South Cove Section B-B
Figure 4.2.1 - Plan View of the Excavation at South Boston: Positions of Geotechnical Instrumentation and Instrumented Sections [Whelan, 1995, p. 87]
Legend: Instrument Types

- IPE
- DMP2
- OW
- INC
- DMP4
- OSPZ
- MPHG
- DMP4

NOTE: Instrument locations are approximate. This figure should not be used for scaling purposes.

Figure 4.2.2 - Plan View of ISS-4 Area: Locations of Building A and Geotechnical Instrumentation [Whelan, 1995, p. 145]
Figure 4.2.3 - Cross-Sectional View of Excavation at ISS-4 Section (approximately at Station 77+20): Locations of tiebacks, Support Walls, and Geotechnical Instrumentation [Whelan, 1995, p. 144]
### Arbed AZ-18 Sheet Piling

<table>
<thead>
<tr>
<th>Profile</th>
<th>$S = \text{Single pce}$</th>
<th>$D = \text{Double pce}$</th>
<th>Mass per ft lb/ft</th>
<th>Sectional area in$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>per $S$</td>
<td>per $D$</td>
<td>per ft of wall</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>50.00</td>
<td>4.69</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>100.00</td>
<td>29.38</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>24.17</td>
<td>7.08</td>
</tr>
</tbody>
</table>

The interlocking joints of AZ-profiles are made for mutual connections. For corner arrangements special connectors are in stock.

### Coating Area

<table>
<thead>
<tr>
<th>Coating area$^1$</th>
<th>Develop. penin.</th>
<th>Section modulus</th>
<th>moment of inertia</th>
<th>Radius of gyration</th>
<th>Profile</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>in</td>
<td>in</td>
<td>lb$^2$/in$^3$</td>
<td>lb$^2$/in$^3$</td>
<td>AZ 18</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$I_y$</td>
<td>$I_x$</td>
<td>per $S$</td>
</tr>
<tr>
<td>5.64</td>
<td>69.3</td>
<td>517.5</td>
<td>870.2</td>
<td>5.93</td>
<td>per $D$</td>
</tr>
<tr>
<td>32.4</td>
<td>138.6</td>
<td>1035.0</td>
<td>5.93</td>
<td>per ft of wall</td>
<td></td>
</tr>
<tr>
<td></td>
<td>33.5</td>
<td>250.4</td>
<td>5.93</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1) Excludes inside of interlocks
2) Considered neutral axe: $y-y$

---

Figure 4.2.4 - Dimensions and Engineering Properties of Arbed AZ-18 Sheet Piling [ISPC, 1990] (section used for South Wall)
Figure 4.2.5a - Selected Effective Stress History Profile at ISS-4 Determined by MIT based on Re-evaluation of the Consolidation Test Data on BBC [Whelan, 1995, p. 130]
Figure 4.2.5b - OCR and Undrained Strength Profiles for Marine Clay throughout the Project Alignment. Compiled from MHD Geotechnical Consultants (1991b) and Special Test Program (MHD Geotechnical Consultant, 1994) [Whelan, 1995, p. 129]
Figure 4.2.6 - Initial In-Situ Stresses Throughout the ISS-4 Soil Profile
[Whelan, 1995, p. 127]
Figure 4.2.7a - Time Period Summary for Step 1: Excavation Geometry on May 7, 1993 [Whelan, 1995, p.192]
Figure 4.2.7b - Time Period Summary for Step 2: Excavation Geometry on June 29, 1993 [Whelan, 1995, p.194]
Figure 4.2.7c - Time Period Summary for Step 3: Excavation Geometry on July 9, 1993 [Whelan, 1995, p.196]
Figure 4.2.7d - Time Period Summary for Step 4: Excavation Geometry on August 9, 1993 [Whelar, 1995, p.198]
Construction Events:
1. 8/11 to 8/16: S
2. 8/16 to 8/17: S
3. 8/20: N
4. 8/24 to 8/28: N
5. 8/28 to 9/2: N

Date: 9/2/1993

Figure 4.2.7e - Time Period Summary for Step 5: Excavation Geometry on September 2, 1993 [Whelan, 1995, p.200]
Figure 4.2.7f - Time Period Summary for Step 6: Excavation Geometry on October 6, 1993 [Whelan, 1995, p.202]
Figure 4.2.7g - Time Period Summary for Step 7: Excavation Geometry on March 11, 1994 [Whelan, 1995, p.204]
Figure 4.2.7h - Time Period Summary for Step 8: Excavation Geometry on May 31, 1994 [Whelan, 1995, p.206]
Figure 4.2.7i - Excavation Schedule as Documented by Whelan (1995) at South Boston ISS-4
Figure 4.2.8 - Locations of Dewatering and Pressure Relief Wells [Whelan, 1995, p.70]
Figure 4.2.9 - Piezometric Pressures in the Lower Aquifer vs. Time
Measured by Four Deep Piezometers Located below the BBC
[Whelan, 1995, p. 180]
South Boston ISS-4

North Diaphragm Wall

61.0-m [200.0-ft]

South Sheet Pile Wall

El. 110'

Cohesive Fill

Organic Deposits

El. 71.5'

Boston Blue Clay (BBC)

El. 7'

8-4 mixed elements for soil

8-noded solid elements for the sheetpile wall, diaphragm wall, and dry soil

1-D spring elements for tiebacks

(See Figure 4.2.11 for locations of tiebacks)

Figure 4.2.10 - Finite Element Mesh used for the Analysis of Excavation at South Boston ISS-4
Step 1S: Excavate Tier 1 South

North Diaphragm Wall

Step 1N: Excavate Tier 1 North and Install 1-S Tiebacks

Step 2: Excavate Tier 2 and Lock-off 1N and 1S Tiebacks

Figure 4.2.11a - Construction Sequence incorporated in the Base Case Analysis at Steps 1S, 1N, and Step 2
Step 3: Excavate Tier 3, Lock-off 2-N Tiebacks, Install 2-S Tiebacks

Step 4: Excavate Tier 4, Lock-off 2-S Tiebacks

Step 5: Excavate Tier 5, Lock-off 3-NTiebacks, Install 3-S Tiebacks

Figure 4.2.11b - Construction Sequence incorporated in the Base Case Analysis at Steps 3, 4 and 5
Step 6: Excavate Tier 6  
Lock-off 3-S Tiebacks

Step 7: Excavate Tier 7

Figure 4.2.11c - Construction Sequence incorporated in the Base Case Analysis at Steps 6 and 7
Figure 4.2.12 - Piezometric Elevations vs. Time, Measured by Four Deep Piezometers in Lower Aquifer vs. Imposed Piezometric Elevations
Figure 4.2.13 - Wall Deflections and Surface Settlements at Construction Step IS (May 7 to June 5, 1993), Base Case Analysis versus Field Measurements
Figure 4.2.14 - Wall Deflections and Surface Settlements at Construction Step 1N [Mar. 22 to May 7, 1993], Base Case Analysis versus Field Measurements
Figure 4.2.15 - Wall Deflections and Surface Settlements at Construction Step 2 [June 5 to July 1, 1993], Base Case Analysis versus Field Measurements
Figure 4.2.16 - Wall Deflections and Surface Settlements at Construction Step 3 [July 1 to Aug. 9, 1993], Base Case Analysis versus Field Measurements
Figure 4.2.17 - Wall Deflections and Surface Settlements at Construction Step 4 [Aug. 9 to Aug. 20, 1993], Base Case Analysis versus Field Measurements
Figure 4.2.18 - Wall Deflections and Surface Settlements at Construction Step 5 [Aug. 20 to Sept. 15, 1993], Base Case Analysis versus Field Measurements
Figure 4.2.19 - Wall Deflections and Surface Settlements at Construction Step 6 [Sept. 15, 1993 to Mar. 2, 1994], Base Case Analysis versus Field Measurements
Figure 4.220 - Wall Deflections and Surface Settlements at Construction Step 7 [Mar. 4 to Mar. 11, 1994]. Base Case Analysis versus Field Measurements.
Figure 4.2.21 - Piezometric Elevations vs. Time, Measured by OW-016 and OW-002 (North Side of Excavation) vs. Base Case Analysis
Figure 4.2.22 - Piezometric Elevations vs. Time, Measured by VWPZ-67 and VWPZ-68 (North Side of Excavation) vs. Base Case Analysis
Figure 4.2.26 - Piezometric Elevations vs. Time, Measured by VWPZ-131 and VWPZ-132 (Inside of Excavation) vs. Base Case Analysis
Figure 4.2.27 - Heave in Clay vs. Time, Measured by MPHG-110 (Inside of Excavation) vs. Base Case Analysis
Figure 4.2.28 - Heave in Clay vs. Time, Measured by MPHG-109 (Inside of Excavation) vs. Base Case Analysis
Figure 4.2.29 - Heave in Clay vs. Time, Measured by MPHG-501 (Inside of Excavation) vs. Base Case Analysis
Figure 4.2.30 - Heave in Clay vs. Time, Measured by MPHG-107 (Inside of Excavation) vs. Base Case Analysis
Figure 4.2.31 - Wall Deflections vs. Time, Measured by INC-102 (North Diaphragm Wall) vs. Base Case Analysis
Figure 4.2.32 - Wall Deflections vs. Time, Measured by INC-101 (South Sheetpile Wall) vs. Base Case Analysis
Figure 4.2.33 - Wall Deflections vs. Time, Measured by IPE-113 (behind South Sheetpile Wall) vs. Base Case Analysis
Figure 4.2.34 - Surface Settlements vs. Time, Measured at Settlement Points behind North Diaphragm Wall vs. Base Case Analysis
Figure 4.2.36 - Settlements vs. Time, Measured at IPE-113 (behind South Sheetpile Wall) vs. Base Case Analysis
Note: 2-S Tieback on the South wall is omitted in the Revised Case Analysis. Glaciomarine, Till, and Bedrock are modeled using EP-DP soil mode.

Figure 4.2.37 - Revised Case FE Analysis including Effects of (1) Underlying Till and Bedrock; (2) Revised Permeability; and (3) Omission of 2-S Tiebacks
Figure 4.2.38 - Wall Deflections and Surface Settlements at Construction Step 1N [Mar. 22 to May 7, 1993], Revised Case Analysis versus Field Measurements
Figure 4.2.39 - Wall Deflections and Surface Settlements at Construction Step 2 [June 5 to July 1, 1993], Revised Case Analysis versus Field Measurements
Figure 4.2.40 - Wall Deflections and Surface Settlements at Construction Step 3 [July 1 to Aug. 9, 1993], Revised Case Analysis versus Field Measurements
Figure 4.2.41 - Wall Deflections and Surface Settlements at Construction Step 4 [Aug. 9 to Aug. 20, 1993], Revised Case Analysis versus Field Measurements
Figure 4.2.42 - Wall Deflections and Surface Settlements at Construction Step 5 [Aug. 20 to Sept. 15, 1993], Revised Case Analysis versus Field Measurements
Figure 4.2.43 - Wall Deflections and Surface Settlements at Construction Step 6 [Sept. 15, 1993 to Mar. 2, 1994], Revised Case Analysis versus Field Measurements
Figure 4.2.44 - Wall Deflections and Surface Settlements at Construction Steps 1N, 3, 5, and 7 Predicted by the Revised Case and Revised Case Analyses
Figure 4.2.45 - Piezometric Elevations vs. Time, Measured by OW-016 and OW-002 (North Side of Excavation) vs. Revised Case Analysis
Figure 4.2.46 - Piezometric Elevations vs. Time, Measured by VWPZ-67 and VWPZ-68 (North Side of Excavation) vs. Revised Case Analysis
Figure 4.2.47 - Piezometric Elevations vs. Time, Measured by OSPZ-106 (South Side of Excavation) vs. Revised Case Analysis
Figure 4.2.48 - Piezometric Elevations vs. Time, Measured by VWPZ-135 and VWPZ-136 (Inside of Excavation) vs. Revised Case Analysis
Figure 4.2.50 - Piezometric Elevations vs. Time, Measured by VWPZ-131 and VWPZ-132 (Inside of Excavation) vs. Revised Case Analysis
Figure 4.2.51 - Heave in Clay vs. Time, Measured by MPHG-110 (Inside of Excavation) vs. Revised Case Analysis
Figure 4.2.52 - Heave in Clay vs. Time, Measured by MPHG-109 (Inside of Excavation) vs. Revised Case Analysis
Figure 4.2.54 - Heave in Clay vs. Time, Measured by MPHG-107 (Inside of Excavation) vs. Revised Case Analysis
Figure 4.2.55 - Wall Deflections vs. Time, Measured by INC-102 (North Diaphragm Wall) vs. Revised Case Analysis
Figure 4.2.56 - Wall Deflections vs. Time, Measured by INC-101 (South Sheetpile Wall) vs. Revised Case Analysis
Figure 4.2.57 - Wall Deflections vs. Time, Measured by IPE-113 (behind South Sheetpile Wall) vs. Revised Case Analysis
Figure 4.2.58 - Surface Settlements vs. Time, Measured at Settlement Points behind North Diaphragm Wall vs. Revised Case Analysis
Figure 4.2.59 - Surface Settlements vs. Time, Measured at Settlement Points behind South Sheeppile Wall vs. Revised Case Analysis
Figure 4.2.60 - Settlements vs. Time, Measured at IPE-113 (behind South Sheepfields Wall) vs. Revised Case Analysis.
Figure 4.3.2 - Stress History Profile for BBC, Based on Data from South Boston Special Test Site [Original Profile]
Figure 4.3.3 - MIT-E3 Representation of Undrained Shear Strength for BBC [Original Profile]
Figure 4.3.4 - Comparison of MIT-E3 [Original Profile] and SHANSEP Strength Profile for BBC
Figure 4.3.5  - Comparison of Field Vane and Undrained DSS Strengths for Original Profile [Revised Profile]
Figure 4.3.6 - Comparison of MIT-E3 [Revised Profile] and SHANSEP Strength Profiles for BBC
Figure 4.3.8 - Excavation Sequence for Platform Section with Excavation Geometry P1
[h = 10.0-ft, toe of DW @ El. 24-ft]
Transitway Platform Section, Excavation Geometry P2

Figure 4.3.9 - Excavation Sequence for Platform Section with Excavation Geometry P2
[h= 13.0-ft, toe of DW @ El. 24-ft]
Transitway Platform Section, Excavation Geometry P3

Figure 4.3.10 - Excavation Sequence for Platform Section with Excavation Geometry P3
\[ h = 13.0\text{-ft, toe of DW @ El. 34-ft} \]
Transitway Transition Section

Figure 4.3.12 - Excavation Sequence for the Transitway Transition Section (T1)
Figure 4.3.13 - Excavation Sequence for Transitway West Tunnel Section (W1)
Figure 4.3.14 — Typical Finite Element Mesh used to Model MBTA Transitway Excavation
Figure 4.3.16 - Ground Surface Displacements for the Transitway Platform Section, Excavation Geometry P1, with Original Profile and Undrained Analysis [P1-O-UD]
Figure 4.3.17 - Bending Moments in the Diaphragm Wall for Transway Platform Section, Excavation Geometry P1, with Original Profile and Undrained Analysis [P1-O-UD]
Figure 4.3.18 - Wall Displacement for the Transitway Platform Section, Excavation Geometry P2, with Original Profile and Undrained Analysis [P2-O-UD]
Figure 4.3.19 - Ground Surface Displacements for the Transitway Platform Section, Excavation Geometry P2, with Original Profile and Undrained Analysis [P2-O-UD]
Figure 4.3.20 - Bending Moments in the Diaphragm Wall for Transitway Platform Section, Excavation Geometry P2, with Original Profile and Undrained Analysis [P2-O-UD]
Figure 4.3.21 - Maximum Wall Deflection, Surface Settlement, and Surface Horizontal Displacement from P1-O-UD and P2-O-UD: Strut Spacing Effects
Figure 4.3.22 - Maximum Diaphragm Wall Bending Moments Above and Below the Lowest Level Strut from P1-O-UD and P2-O-UD: Strut Spacing Effects, Platform Section
Figure 4.3.23 - Strut Loads for Transitway Platform Section with Excavation Geometries P1 and P2, Undrained Analysis and Original Profile [P1-O-UD and P2-O-UD]
Figure 4.3.24 - Wall Displacement for the Transitway Platform Section, Excavation Geometry P3 with Original Profile and Undrained Analysis [P3-O-UD]
Figure 4.3.25 - Ground Surface Displacements for the Transitway Platform Section, Excavation Geometry P3, with Original Profile and Undrained Analysis [P3-O-UD]
Figure 4.3.26 - Bending Moments in the Diaphragm Wall for Transitway Platform Section, Excavation Geometry P3, with Original Profile and Undrained Analysis [P3-O-UD]
Figure 4.3.27 - Maximum Wall Deflection, Surface Settlement, and Surface Horizontal Displacement from P2-O-UD and P3-O-UD: Wall Length Effects
Figure 4.3.28 - Maximum Diaphragm Wall Bending Moments Above and Below the Lowest Level Strut from P2-O-UD and P3-O-UD: Wall Length Effects, Platform Section
Figure 4.3.29 - Strut Loads for Transitway Platform Section with Excavation Geometries P2 and P3, Undrained Analysis and Original Profile [P2-O-UD and P3-O-UD]
Figure 4.3.30 - Wall Displacement for the Transitway Platform Section, Excavation Geometry P1, with Original Profile and Partially Drained Analysis [P1-O-PD]
Figure 4.3.31 - Ground Surface Displacements for the Transitway Platform Section, Excavation Geometry P1, with Original Profile and Partially Drained Analysis [P1-O-PD]
Figure 4.3.32 - Bending Moments in the Diaphragm Wall for Transitway Platform Section, Excavation Geometry P1, with Original Profile and Partially Drained Analysis [P1-O-PD]
Figure 4.3.33 - Maximum Wall Deflection, Surface Settlement, and Surface Horizontal Displacement from P1-O-UD and P1-O-PD: Partial Drainage Effects
Figure 4.3.34 - Maximum Diaphragm Wall Bending Moments Above and Below the Lowest Level Strut from P1-O-UD and P1-O-PD: Partial Drainage Effects, Platform Section
Figure 4.3.35 - Strut Loads for Transitway Platform Section with Excavation Geometries P1, P1-O-UD and P1-O-PD: Partial Drainage Effects
Figure 4.3.37 - Ground Surface Displacements for the Transitway Platform Section, Excavation Geometry P3, with Original Profile and Partially Drained Analysis [P3-O-PD]
Figure 4.3.38 - Bending Moments in the Diaphragm Wall for Transitway Platform Section, Excavation Geometry P3, with Original Profile and Partially Drained Analysis [P3-O-PD]
Figure 4.3.39 - Maximum Wall Deflection, Surface Settlement, and Surface Horizontal Displacement from P3-O-UD and P3-O-PD: Partial Drainage Effects
Figure 4.3.40 - Maximum Diaphragm Wall Bending Moments Above and Below the Lowest Level Strut from P3-O-UD and P3-O-PD: Partial Drainage Effects, Platform Section
Figure 4.3.41 - Strut Loads for Transitway Platform Section with Excavation Geometries P3, P3-O-UD and P3-O-PD: Partial Drainage Effects
Figure 4.3.42 - Wall Displacement for the Transitway Platform Section, Excavation Geometry P3 with Original and Revised Profiles, Partially Drained Analysis [P3-O-PD and P3-R-PD]
Figure 4.3.43 - Ground Surface Displacements for the Transitway Platform Section, Excavation Geometry P3, P3-O-PD and P3-R-PD
Figure 4.3.44 - Bending Moments in the Diaphragm Wall for Transitway Platform Section, Excavation Geometry P3, with Revised Profile and Partially Drained Analysis [P3-R-PD]
Figure 4.3.45 - Maximum Wall Deflection, Surface Settlement, and Surface Horizontal Displacement from P3-O-PD and P3-R-PD: Stress History Effects
Figure 4.3.46 - Maximum Diaphragm Wall Bending Moments Above and Below the Lowest Level Strut from P3-O-PD and P3-R-PD.
Figure 4.3.47 - Strut Loads for Transitway Platform Section with Excavation Geometry P3, Partially Drained Analysis with Original and Revised Profiles [P3-O-PD & P3-R-PD]
Transitway Mezzanine Section [M1]

- **h = 13-ft**

- PZ-38 Sheetpile Wall
- 0.9-m Concrete Diaphragm Wall

**Figure 4.3.48 - Transitway Mezzanine Section Cross-sectional View**
Figure 4.3.49 - Wall Displacement for the Transitway Mezzanine Section with Original and Revised Profiles, Partially Drained Analysis [M1-O-PD and M1-R-PD]
Figure 4.3.50 - Ground Surface Displacements for the Transitway Mezzanine Section, with Original and Revised Profiles and Partially Drained Analysis [M1-O-PD and M1-R-PD]
Figure 4.3.51 - Bending Moments in the Diaphragm Wall for Transitway Mezzanine Section, Original and Revised Profiles and Partially Drained Analysis [M1-O-PD and M1-R-PD]
Figure 4.3.52 - Maximum Wall Deflection, Surface Settlement, and Surface Horizontal Displacement from M1-O-PD and M1-R-PD: Mezzanine Section Analysis Summary
Figure 4.3.53 - Maximum Diaphragm and Sheetpile Wall Bending Moments Above and Below the Lowest Level Strut from M1-O-PD and M1-R-PD: Mezzanine Section
Figure 4.3.54 - Strut Loads for Transitway Mezzanine Section with
Original and Revised Profiles, Partially Drained Analysis
[M1-O-PD and M1-R-PD]
Figure 4.3.56 - Wall Displacement and Moment in the Diaphragm Wall for the Transition Section with Original and Revised Profiles and Partially Drained Analysis [T1-O-PD and T1-R-PD]

- Wall Displacement (in)
  - T1-O-PD
  - T1-R-PD

- Depth (m)
  - h = 13-ft
  - Toe of Dia. Wall @ El. 24-ft
  - Partially Drained:
    - 45 days for Steps 1 to 4
    - 135 days for Step 5

- Wall Displacement (cm)

- Bending Moment (kips-ft/ft)

- Bending Moment (MN·m/m)

- Elevation (ft)
Figure 4.3.57 - Ground Surface Displacements for the Transitway Transition Section, with Original and Revised Profiles and Partially Drained Analysis [T1-O-PD and T1-R-PD]
Figure 4.3.58 - Maximum Wall Deflection, Surface Settlement, and Surface Horizontal Displacement from T1-O-PD and T1-R-PD: Transition Section Analysis Summary
Figure 4.3.59 - Maximum Diaphragm Wall Bending Moments Above and Below the Lowest Level Strut from T1-O-PD and T1-R-PD: Transition Section
Figure 4.3.60 - Strut Loads for Transitway Transition Section with Original and Revised Profiles, Partially Drained Analysis [T1-O-PD and T1-R-PD]
Figure 4.3.61 - Transitway West Tunnel Section Cross-sectional View

Distance from Centerline (m)

Depth (m)

0.9-m Concrete Diaphragm Wall

Transitway West Tunnel Section [W1]

h = 13-ft

Till / Bedrock

Boston Blue Clay

Silty Sand

Cohesive Fill

Distance from Centerline (ft)

Elevation (ft)
Figure 4.3.62 - Wall Displacement and Moment in the Diaphragm Wall for the West Tunnel Section with Original and Revised Profiles and Partially Drained Analysis [W1-O-PD and W1-R-PD]
Figure 4.3.63 - Ground Surface Displacements for the Transitway
West Tunnel Section, with Original and Revised Profiles
and Partially Drained Analysis [W1-O-PD and W1-R-PD]
Figure 4.3.64 - Maximum Wall Deflection, Surface Settlement, and Surface Horizontal Displacement from W1-O-PD and W1-R-PD: West Tunnel Section Analysis Summary
Figure 4.3.65 - Maximum Diaphragm Wall Bending Moments Above and Below the Lowest Level Strut from W1-O-PD and W1-R-UD: West Tunnel Section
Figure 4.3.66 - Strut Loads for Transitway West Tunnel Section with Original and Revised Profiles, Partially Drained Analysis [W1-O-PD and W1-R-PD]
Figure 4.3.67 - Summary of Maximum Displacements from the Four Typical Sections in the Transitway Project: Width Effects
CHAPTER 5

Prediction of Surface Settlements for Excavations in Clay-Dominated Profiles

Surface settlements are usually considered the best indicator for the performance of an excavation because the magnitude and the distribution of the settlement can be directly related to potential damage to adjacent structures. As a result, accurate and reliable predictions of the surface settlements are primary concern in design, especially for excavations with sensitive adjacent facilities. The existing methods used to estimate excavation-induced settlements are briefly summarized in Chapter 2. These existing techniques provide very little guidance for estimating settlements where the computed factor of safety against basal heave is less than 1.5. The methods recommended in this chapter apply for excavations in clay-dominated profiles where soft clay underlies the excavation (and hence, FS is often less than about 1.5). The proposed methods, presented in this Chapter, are based on the results of numerical experiments presented in Chapter 3. These methods estimate the isolated effects of excavation geometry (Section 3.2), soil profile (Section 3.3), and support structure (Section 3.4).
Section 5.1 summarizes the proposed method for predicting the surface settlements for plane strain, undrained excavations in normally consolidated BBC supported by 0.9-m thick concrete diaphragm wall, with rigid supports at a vertical spacing, \( h = 2.5\text{-m} \) (DW-RS). This recommendation, based on results from Section 3.2, includes effects of wall length, \( L \), depth to bedrock, \( d_B \), and excavation width, \( B \). Section 5.2 extends the method for excavations in other clay-dominated soil profiles including the presence of a cohesionless stratum of depth, \( d_s \), and overconsolidated clay crust above a lower deposit of the soft clay (using results from Section 3.3). Section 5.3, based on results presented in Section 3.4, includes the effects of support structure stiffness. The impact of changes in support stiffness on the surface settlement can be quantified as a function of overall support stiffness and the soil profile near the excavation grade. A summary and limitation of the recommendations is given in Section 5.4.

5.1 Predictions of Settlement for Excavations in Normally Consolidated BBC with various Excavation Geometries

The numerical experiments evaluating the effects of changes in excavation geometries in normally consolidated clay are presented in Section 3.2. As shown in Figure 5.1.1, the typical shape of the settlement trough consists of a "trough" and a "tail" separated by a point of inflexion marked I. The location of the trough, \( x_{(\text{max})} \), is determined by the length of the wall, the excavation depth, and to some extent, the excavation width (i.e., \( L \), \( H \), and \( B \)). The tail of the settlement is highly dependent on the location of the bedrock relative to the excavation depth (i.e. \( d_B : H \)). Detailed descriptions of the influence of each parameter are presented in Section 3.2; however, Figure 3.2.31 illustrates the basic effects of changes in excavation geometry.
Based on the results from the numerical experiments and an extensive program of regression analyses, the surface settlement for any given geometry for excavation depth, $H \geq 7.5$-m can be described by the following dimensional equation (Eq. 5.1.1):

$$\delta_v = \alpha \left[ e^{(ax^2 + bx)} \right] (1 + x^2)^c$$  \hspace{1cm} (5.1.1)

where 
- $\delta_v$ \hspace{1cm} settlement [cm] 
- $x$ \hspace{1cm} distance from wall [m] 
- $\alpha$, $a$, $b$, $c$ \hspace{1cm} coefficients dependent on the excavation geometry

With proper values of $\alpha$, $a$, $b$, and $c$, Equation 5.1.1 is capable of describing the trough and tail of the surface settlement obtained from the numerical analyses. The normalized settlements, shown in Figure 3.2.31, can be described by dividing equation 5.1.1 by the maximum surface settlement, $\delta_{v(max)}$, thus yielding Equation 5.1.2. This normalized settlement curve consists of 3 coefficients ($a$, $b$, and $c$) with a maximum value of 1.

$$\frac{\delta_v}{\delta_{v(max)}} = \frac{\left[ e^{(ax^2 + bx)} \right] (1 + x^2)^c}{\beta}$$  \hspace{1cm} (5.1.2)

where 
- $\delta_{v(max)}$ \hspace{1cm} maximum settlement in cm 
- $\beta = \frac{\delta_{v(max)}}{\alpha}$ \hspace{1cm} maximum value of 
  \hspace{1cm} $e^{(ax^2 + bx)(1 + x^2)^c} = f(a, b, and c)$ 
  \hspace{1cm} (see Figure 5.1.2)

The recommended method for predicting surface settlements for excavations in normally consolidated clay consists of two components for

---

124 The intent of the regression analyses is to determine a family of curves which best describe the surface settlements. Equation 5.1.1 was the simplest equation, in term of number of coefficients, which captures all aspects of the surface settlement trough.
every excavation geometry: 1) predict the normalized settlement, and 2) predict the maximum surface settlement. Section 5.1.1 describes proposed procedures for obtaining this normalized surface settlement, as well as illustration of its capabilities. Section 5.1.2 presents the recommended method for estimating $\delta_{v(\text{max})}$ and thus, thus defining the entire profile of surface settlement.

5.1.1 Normalized Surface Settlement

Equation 5.1.2 is capable of describing the normalized settlements presented in Figure 3.2.31. The value of $\beta$, which is a function of $a, b,$ and $c$, can be obtained based on the equations outlined in Figure 5.1.2. Figure 5.1.3 presents the relationships between the values of the coefficients ($a, b,$ and $c$) and the geometric parameters ($B, L, d_B,$ and $H$). These relationships were derived by performing a second set of regression analyses attempting to relate the coefficients ($a, b,$ and $c$) derived from the first set of regression analyses with the geometric parameters. Figure 5.1.3 represents the final result of this process. As shown in Figure 5.1.3, one additional factor, $d_B^*$, is included in the calculation of $a, b,$ and $c$. This additional parameter is the adjusted depth to bedrock to account for the relative magnitudes of excavation width and wall length. Calculation of $d_B^*$ for any given geometry is described in Figure 5.1.4.

Figure 5.1.3 shows how each geometric parameter influence the shape of the surface settlement based on how they influence the coefficients. The results can be summarized as the following:

i. coefficient $a$, which influence the tail portion of the settlement, depends on the value of the depth to bedrock ($d_B$) and its relative magnitude with respect to wall length ($L$) and excavation width ($B$).
ii. coefficient $b$ consists of two components: $b'$ and $b^*$. The component $b'$ reflects the wall deformation mode by including the excavation depth (H) and the wall length (L); while $b^*$ relates to the depth to bedrock. For cases where $d_B^* \geq 3.5H$, the thicker underlying clay layer has negligible impact on the surface settlements; however, if $d_B^* < 3.5H$, the close proximity of the underlying rigid base can influence the trough portion of the surface settlement.

iii. coefficient $c$ also has two components: $c'$ representing the wall deformation mode shape and $c^*$ describing the effect of the rigid base. The location of the trough is captured by coefficients $a$, $b$ and $c$ as described in Figure 5.1.2.

Figure 5.1.5 summarizes this recommended procedure for predicting the normalized surface settlements. However, this method is limited to estimating the surface settlements ($H \geq 7.5$-m) for undrained excavations in normally consolidated BBC supported by 0.9-m thick concrete diaphragm wall with rigid supports at $h = 2.5$-m. This procedure is capable of capturing the surface settlements from all the excavation problems listed in Table 3.2.2. Section 5.2 extends this method to include different soil profiles and Section 5.3 generalizes this prediction method for support conditions other than diaphragm walls with perfect struts.

For a given set of geometric parameters $B$, $L$, $d_B$, and $H^{125}$, the normalized surface settlement can be estimated following 5 steps:

1. Define magnitudes of $B$, $L$, $d_B$, and $H$.
2. Using Figure 5.1.4, determine the adjusted depth to bedrock ($d_B^*$).

---

125 The proposed design method is applicable for $H \geq 7.5$-m, $15m \leq B \leq 80$m, $25m \leq L \leq 40$m, and $1.2L \leq d_B \leq 100$m.
3. Use Figure 5.1.3 to calculate values of $a$, $b'$, $b^*$, $c'$, and $c^*$.
4. Use Figure 5.1.2 to calculate value of $\beta$.
5. Determine the normalized settlement trough, $\delta_V/\delta_{V(\text{max})}$, by substituting the coefficients obtained in steps 3 and 4 into Equation 5.1.2.

Three illustrations of method outlined in Figure 5.1.5 are shown in Figures 5.1.6a to 5.1.6c. Figure 5.1.6a compares the normalized settlement troughs computed by the empirical equations (Figure 5.1.5) with results from eight finite element finite element analyses of 80-m wide excavations ($B = 80\text{m}$) at an excavation depth of $H = 15.0\text{m}$. Three of these eight analyses are supported by 40-m long wall ($L = 40\text{m}$) with three depths to bedrock ($d_B = 50\text{m}$, 75m, and 100m). The remaining five are supported by 25-m long wall ($L = 25\text{m}$) with five possible depths to bedrock ($d_B = 30\text{m}$, 37.5m, 50m, 75m, and 100m). The dashed lines represent the finite element results for the given geometry and solid lines are the predicted normalized surface settlements following procedures outlined in Figure 5.1.5. Figures 5.1.6b and 5.1.6c presents similar results from two other sets of analyses: one from 40-m wide excavations and the other from 20-m wide excavations. In general, excellent predictions can be obtained for intermediate to wide excavations. For narrow excavations with shallow bedrock, the predictions are in good agreement with the numerical results; however, the predicted normalized settlement tend to overestimate the tail portion of the settlement troughs for cases when the bedrock is two times greater than the excavation width.

5.1.2 Maximum Surface Settlement

The previous section describes procedures for estimating the normalized surface settlement. This section completes the prediction of settlements for excavations in normally consolidated BBC by providing
design charts for estimating the maximum surface settlement, $\delta_v(\text{max})$. Figure 5.1.7 summarizes the maximum surface settlements for $H \geq 7.5$-m from 24 numerical excavations supported by 40-m long wall and 40 excavations supported by 25-m long walls covering a wide range of excavation widths ($B$) and depth to bedrock ($d_B$). As stated in Section 3.2, the magnitudes of the surface settlement increase approximately bi-linearly with excavation width at an increasing rate as the excavation progresses. For a given excavation width, the maximum surface settlements generally increase as the location of the bedrock becomes shallower.

This observed behavior of maximum surface settlement in response to changes in the excavation width can be described by the following equation:

$$\delta^*_v(\text{max}) = \left( \frac{B}{2m} + 1 \right) \left[ i - \left( H^2 - 7.5^2 \right) j \right]$$  \hspace{1cm} (5.1.3)

where $\delta^*_v(\text{max})$ maximum surface settlement without accounting for $d_B$ effects (in cm),
$B$ excavation width in meters,
$H$ excavation depth in meters,
$m$, $i$, & $j$ coefficients dependent on $L$ and $B$
(see Figure 5.1.8)

The predicted maximum surface settlements based on this Equation 5.1.3 are also included in Figure 5.1.7. Figure 5.1.8 includes the recommended values of $m$, $i$, and $j$ for any given combination of excavation width ($B$) and wall length ($L$). Comparison with the finite element results (Figure 5.1.7) indicates that Equation 5.1.3 describes the basic trend of maximum surface settlement as a function of excavation width ($B$), excavation depth ($H$), and wall length ($L$). The effect of the bedrock depth can then be included through a simple adjustment factor, $\mu$, in order to calculate the final settlement:

$$\delta_v(\text{max}) = \mu \delta^*_v(\text{max})$$  \hspace{1cm} (5.1.4)
where \( \delta_{v(\text{max})} \) maximum surface settlement in cm (NC BBC)
\( \delta^*_{v(\text{max})} \) from Equation 5.1.3
\( \mu \) bedrock adjustment factor (see Figure 5.1.8)

5.1.3 Prediction of Surface Settlement for Excavations in NC BBC

Sections 5.1.1 and 5.1.2 provide a methodology for estimating the surface settlements for an excavation in NC BBC supported by 0.9-m concrete diaphragm wall and rigid supports with \( H \geq 7.5 \text{m}, 15 \text{m} \leq B \leq 80 \text{m}, 25 \text{m} \leq L \leq 40 \text{m}, \) and \( 1.2L \leq d_B \leq 100 \text{m} \). Figures 5.1.9a and 5.1.9b illustrate the predictive capabilities of this recommended procedure by comparing predictions with results from two finite element analyses. The coefficients for describing the distribution of the surface settlements (Equation 5.1.2) and the coefficients for calculating the maximum surface settlement (Equation 5.1.3) are also shown in the figures. In both cases, the proposed prediction method yields a very accurate representation of the computed settlement troughs (within \( \pm5\% \) of the finite element results at all excavation depths, \( H \), and distances, \( x \)).

5.2 Predictions of Settlement for Various Soil Profiles

Section 5.1 focuses on the behavior of excavations in soil profile consisting of only normally consolidated BBC (as presented in Section 3.2). Section 3.3 presents more comprehensive studies showing the effects of the soil profile on excavation performance. The goal of this section is to expand the previous equations for estimating settlement troughs in these more complex clay-dominated profiles which include overlying non-clay materials.

The recommended procedures presented in this section are based on Groups D, E, and F numerical experiments presented in Section 3.3; therefore, are applicable for excavations in soils with significant amount of underlying soft clay supported by 0.9-m thick concrete diaphragm wall (DW) with rigid
supports (RS). Section 5.2.1 describes how changes in the soil profile impacts the distribution of surface settlement. The influence of soil profile on the magnitude of the surface settlement is addressed in Section 5.2.2. Section 5.2.3 combines the results from Sections 5.2.1 and 5.2.2 by presenting the final predicted surface settlement.

5.2.1 Distribution of Surface Settlement in a Composite Profile

Section 3.3 contains results from three extensive sets of numerical analyses examining the effects of changes in soil stress history profile (Group D), the presence of a cohesionless layer (Group E), and the clay crust (Group F). The results from these numerical experiments show that the cohesionless layer and the clay crust generally reduces the magnitude of the surface settlement within the "trough" as well as moving the "trough" further away from the excavation (see Figures 3.3.37, 3.3.39, and 3.3.41). However, the cohesionless layer and the clay crust appear to have negligible impact on the tail portion of the settlement. This observation suggests that far field deformations are dependent on the underlying clay rather than the soil strata near the surface. Consequently, the procedures presented in Section 5.1, for NC BBC, are adequate for predicting settlement at a distance from the excavation; The required adjustments on the trough portion of the settlement are discussed in Section 5.2.2.

5.2.2 Magnitude of Maximum Surface Settlement in a Composite Profile

As discussed in Section 3.3, changes in the stress history in the clay profile have a tremendous impact on the magnitude of soil movements (see Group D results). Figure 5.2.1 summarizes the maximum surface settlements obtained from the Group D analyses as a function of clay OCR and the
excavation depth. Slight deviations from the normally consolidated state result in dramatic reduction in the surface settlement. For normally consolidated BBC, the ratio between the maximum surface settlement and the excavation depth \( (\delta_v(\text{max})/H) \) is highly dependent on the excavation depth; however, for OCR greater than 2, this ratio remains constant independent of H (see Figure 3.3.11). Based on this observation, the recommended method focuses on simulating the behavior of lightly overconsolidated BBC (OCR \leq 1.4). The maximum surface settlement for OCR's other than 1 can be approximated by the following equation:

\[
\delta_v^{\ast}(\text{max}) = \left[1 + \frac{B}{2m}\left(\frac{i}{OCR} - \left(H^2 - 7.5^2\right)\frac{j}{OCR^2}\right)\right]
\]  

(5.2.1)

where

- OCR overconsolidation ratio
  - for OCR \leq 1.4: use actual OCR
  - for OCR > 1.4: use OCR = 1.4
- B excavation width in meters
- H excavation depth in meters
- m, i, & j coefficients (see Figure 5.1.8)

Figure 5.2.1 compares the predictions from Equation 5.2.1 with finite element calculations of \( \delta_v(\text{max}) \). The proposed equation provides a good representation of results for OCR \leq 1.4; however, it tends to overestimate the settlements for more overconsolidated profiles, especially at larger H.

The presence of an overlying cohesionless layer also affects the magnitude of the surface settlement (see Group E results in Section 3.3). Figure 5.2.2 summarizes this effect by plotting the ratio between the maximum settlement with the cohesionless layer and the maximum settlement in an all-clay profile \( (\lambda = \delta_v(\text{max})\text{ with cohesionless} / \delta_v(\text{max})\text{ all clay}) \) as a function of the difference between the thickness of the cohesionless layer and the excavation depth \([d_s -(H+2.5)]\). If the cohesionless soil / clay interface is at
2.5-m below the excavation grade (i.e. \([d_s - (H+2.5)] = 0\)), then the maximum settlement is unaffected by the cohesionless layer. At excavation depths that are significantly above the cohesionless soil / clay interface (i.e. \([d_s - (H+2.5)] > 0\)), then a reduction in the maximum surface settlement is expected. If the excavation is below this interface (i.e. \([d_s - H] < 0\)), then the maximum settlement is expected to increase by approximately 10%. Figure 5.2.2 includes a function for \(\lambda\) which quantifies these effects.

Figures 5.2.1 and 5.2.2 summarize the independent effects of changes in clay stress history and the presence of an overlying cohesionless layer. Figure 5.2.3 illustrates how these effects can be combined to estimate the impact of a composite profile which includes a cohesionless layer and a decreasing OCR with depth profile. This procedure consists of 4 steps:

1. determine the equivalent OCR\(^{126}\) by averaging the value of the OCR at the toe of the wall and the average OCR over zone A.
2. calculate \(\delta^*_{v_{(\text{max})}}\) by substituting this equivalent OCR in Equation 5.2.1.
3. calculate the adjustment factor to account for the location of the bedrock, \(\mu\), and the cohesionless layer, \(\lambda\).
4. obtain the maximum settlement by including the adjustment factors in step 3: \(\delta_{v_{(\text{max})}} = \mu \lambda \delta^*_{v_{(\text{max})}}\).

The performance of this method is also illustrated in Figure 5.2.3 through comparison with results of the Group F numerical experiments (Section 3.3.3). In general, the predictions are in good agreement with the FE results; but the predicted settlements tend to be underestimated at the final excavation depth just prior to failure.

\(^{126}\) If OCR > 1.4, use 1.4 in calculating the average; if OCR ≤ 1.4, the actual value is used in the average.
5.2.3 *Predicted Surface Settlement for Excavations in a Composite Profile*

Figure 5.2.4 illustrates the procedures for estimating the settlements by combining the procedures outlined in Sections 5.1 and 5.2. The predicted settlement is obtained by A) determining the settlement trough for an excavation of the same dimension in normally consolidated BBC (Section 5.1); and B) calculate the maximum surface settlement for the given profile (Section 5.2). The final predicted settlement is the composite of Steps A and B. The predicted settlement distribution is compared with results for Profile F1 (from Figure 3.3.36) at H = 15-m. The predictions are in very good agreement with the results of finite element analysis. Figures 5.2.5a, 5.2.5b, and 5.2.5c illustrate further the predictive capabilities of this method in other profiles and at other excavation depths.

5.3 *Predictions of Settlement for Various Support Systems*

Although the methods presented in Sections 5.1 and 5.2 can be used to estimate settlements for excavations in various types of clay-dominated soil profiles, the predictions are limited to excavations supported by relatively rigid support system comprising a 0.9-m thick concrete diaphragm wall with rigid bracing at vertical spacing, \( h = 2.5 \text{m} \). Most support systems used in practice are more flexible than this assumed system. Therefore, this section provides recommendations for estimating the impact of changes in the support system. All the recommendations are based on the numerical experiments in Groups G and H thus covering a wide range of wall stiffness and strut stiffness; however, the vertical spacing of the strut is assumed to remain unchanged at \( h = 2.5 \text{m} \). Section 5.3.1 discusses the impact of the support system on the distribution of the surface settlement. The influence of
the support system stiffness on the maximum surface settlement is presented in Section 5.3.2. The prediction of surface settlement, accounting for the support system stiffness, is summarized in Section 5.3.3.

5.3.1 Effect of Support System on Distribution of Surface Settlements

Variations in the support system stiffness have negligible impact on the far field settlements. Group G analyses, presented in Section 3.4, show that decreasing the wall stiffness will increase the maximum settlement and cause the trough portion of the settlement to migrate toward the excavations. However, at a distance beyond \( x > 3H \), the distributions and the magnitudes of the surface settlements are unaffected by the support system and are dependent on the underlying soft clay and location of the underlying rigid base.

Group H analyses examine the effects of strut stiffness on the excavation behavior. As summarized in Section 3.4, decreasing the strut stiffness causes an increase in the maximum settlement but with little impact on the location of the trough, and no influence on far field settlements. The amount of increase in the maximum surface settlement in response to changes in the stiffness of the support wall and the struts is discussed and quantified in the next section, Section 5.3.2.

5.3.2 Maximum Surface Settlement for Various Support System Stiffness

Figure 5.3.1 summarizes the surface settlement ratio, \( \omega \), as a function of the support system stiffness. The settlements are normalized with respect to the maximum settlement corresponding to the most rigid support conditions.
used in the analysis (i.e. 0.9-m thick concrete diaphragm wall, \( EI = 1442 \) MN-m\(^2\)/m, with rigid bracings). The support system stiffness\(^{127}\) is defined as

\[
\text{Support System Stiffness} = \text{SSS} = \log \left( \frac{k_s}{300} \frac{EI}{EI} \right)
\]  
\[(5.3.1)\]

where

- \( k_s \) strut stiffness in MN/m/m;
- (for \( k \geq 300 \) MN/m/m, use \( k = 300 \) MN/m/m)
- \( EI \) wall bending stiffness in MN-m\(^2\)/m

Based on this definition of support system stiffness, the results for perfect struts are applicable for \( k_s \geq 300 \) MN/m/m. For normally consolidated BBC profile, a linear relationship exists between the surface settlement ratio and the support system stiffness (except when the excavation is close to failure). The relationship between maximum surface settlement ratio and system stiffness can be described by the following:

\[
\omega = \frac{\delta_{v(\text{max})}}{\delta_{v(\text{max})}/D_{Ww}/RS} = (1 - \pi) \left[ \frac{\log \left( \frac{k_s}{300} \frac{EI}{EI} \right)}{\log(1442)} \right] + \pi
\]  
\[(5.3.2)\]

where

- \( \omega \) normalized settlement
- \( \pi \) support system stiffness adjustment factor, \( f(\text{soil type, OCR}) \) (see Figure 5.3.3)
- \( k_s \) strut stiffness in MN/m/m
- \( EI \) wall bending stiffness in MN-m\(^2\)/m
- \( \delta_{v(\text{max})}/D_{Ww}/RS \) Maximum settlement for same excavation geometry supported by 0.9-m concrete diaphragm wall with rigid bracing (\( h = 2.5\)-m)

The slope of this function, \((1-\pi)/3.159 \)\(^{128}\), is dependent on the soil type and/or stress history in the vicinity of the excavation grade. Figure 5.3.3 summarizes

\(^{127}\) Note that the support system stiffness should also include the vertical support spacing, \( h \). In the current analyses, a constant value of \( h = 2.5\)-m is assumed therefore, this variable \( h \) is not included in this definition.

\(^{128}\) Note, \( 3.159 = \log(1442) = \log(\text{EI for 0.9-m diaphragm wall}) \).
the procedures required for obtaining values of $\pi$ for a given soil type and/or OCR. The value of $\pi$ is dependent on the soil properties between the location of the last strut and 7.5-m below the excavation grade. For the overlying cohesionless layer, a value of $\pi = 2.5$ is appropriate. Within the BBC stratum, $\pi$ is a function of OCR as indicated in Figure 5.3.3.

The overall maximum surface settlement which includes the bedrock effects, soil profile effects, and the support system stiffness effects can be obtained as the following:

$$\delta_{v(max)} = \mu \lambda \omega \left( \delta_{v(max)}^* \right)$$  \hspace{1cm} (5.3.3)

where $\delta_{v(max)}$ is maximum overall settlement [cm]

$\mu$ is factor for bedrock depth (Figure 5.1.8)

$\lambda$ is factor for soil profile (Figure 5.2.2)

$\omega$ is factor for support system stiffness (Figure 5.3.3)

$\delta_{v(max)}^*$ is max. settlement for a given soil stress history [cm] (Figures 5.2.1 and 5.1.8)

Figures 5.3.2a and 5.3.2b demonstrate this procedure for estimating values of $\omega$ for excavations in profiles F1, F2, F3, and F4. Also included in the figures are four to five sets of finite element results for each soil profile. Profiles F1 and F3 consist of five sets of analyses represent observed settlements for excavations supported by (1) 0.9-m thick concrete diaphragm wall with rigid supports; (2) 0.9-m thick concrete diaphragm wall with compressible\textsuperscript{129} struts; (3) PZ-38 sheet pile wall with rigid supports; (4) PZ-38 sheet pile wall with compressible struts; and (5) PZ-27 sheet pile wall with rigid supports. Profiles F2 and F4 consist of only the first four support system described above. In general, the method outlined in Figure 5.3.3 is able to

\textsuperscript{129} "Real" struts have a strut stiffness of $k_s = 74.4$ MN/m/m.

Page 645
capture the basic behavior of the maximum surface settlement as a function of support system stiffness.

5.3.3 Surface Settlement for Various Support System Stiffness

The method used for the prediction of the support system stiffness is very similar to the method described in Section 5.2.3. The tail of the surface settlement is independent of the overlying soil profile and the support structure and therefore, can be estimated from results of excavations in normally consolidated BBC (curve a in Figure 5.3.4). The trough of the surface settlement is defined by the estimated maximum surface settlement, $\delta_{v(max)}$, obtained from Equation 5.3.3 (line b in Figure 5.3.4). Figure 5.3.4 illustrates the calculations of the surface settlement for an excavation in normally consolidated BBC supported by PZ-38 sheet pile wall and compressible struts. As shown in Figure 5.3.4, curve b lies below curve a, the predicted settlement curve can be constructed by connecting the point 1 (located below the maximum point of curve a) to point 2 (located 3H from the wall). Figure 5.3.5 includes another example of similar calculations for an excavation in Profile F1 supported by the same PZ-38 sheet pile wall and the real struts. As shown in these two figures, the recommended method is capable of yielding good predictions of surface settlements for any given excavation geometry, soil profile, and support system.

5.4 Summary of Design Recommendations for Predicting Surface Settlements

The procedures for predicting surface settlements for any given excavation geometry and soil profile are illustrated in Figures 5.3.4 and 5.3.5. These recommendations are based on results from an extensive program of numerical parametric analyses evaluating the effects of excavation geometry,
soil profile, and support system. Detailed descriptions of numerical experiments are presented in Chapter 3; summary of results and general observed trends are incorporated in the design recommendations.

The results of the numerical analysis suggest that surface settlements at a distance from the excavation are dependent on the behavior of the underlying stratum of soft clay. The overlying soil and variations in the support system have minimal effect on this surface settlement tail. In fact, the effects of the overlying soil and variations in the support system are confined to the trough portion of the surface settlement located within 3H from the excavation. Stiffer overlying soil will cause a reduction in the settlement within the trough as well as causing the trough to migrate further from the excavation. A more flexible support system has the opposite effect causing an increase in the maximum settlement and moving its location closer to the excavation support wall.

The recommended procedure, incorporates these general observations in estimating the surface settlement. For any given excavation geometry, soil profile, and support system, the projected settlements within the tail are estimated to be the same as an excavation with the same geometry but in an idealized normally consolidated BBC profile (see Section 5.1). The predicted maximum settlement corresponding to the specific excavation geometry, soil profile, and support structure is use to describe the trough portion of the surface settlement (see Sections 5.2 and 5.3). Comparisons of these predicted settlements with the numerical experimental results show that this recommended procedure is able to describe accurately within 15% of the surface settlements in response to changes in the excavation geometry, soil profile, and support system (see Figures 5.3.4 and 5.3.5).
Using empirical data obtained from past excavations in soft to medium clay, Clough and O'Rourke [1990] determined that the excavation depth (H) is the main parameter in defining the shape of the surface settlement for excavations in soft to medium clay (see Figure 2.2.5). According to their recommendations, the largest settlement should occur within a distance of 0.75H from the excavation; and beyond 2H, the surface settlements should be negligible. However, the findings from the numerical experiments performed in this research suggest that the location of the bedrock relative to the excavation depth (d_B : H) play a significant role in defining the distributions of the surface settlement; and the tail portion of the surface settlement is defined by the underlying soft clay deposit with negligible effects from the overlying soils and the support system. Consequently, distributions of surface settlements satisfying Clough and O'Rourke's description only occur when the d_B << 3.5H and the excavation is supported by a more flexible support system. Therefore, observed settlements for excavations in deeper deposits of clay and supported by a more rigid system are likely to fall outside of this settlement envelope recommended by Clough and O'Rouke.

In terms of maximum surface settlements, the numerical experiments showed that shallower bedrock depth, more flexible support system, excavation extending below the cohesionless stratum, and more normally consolidated clay deposit will cause an increase in the settlement. However, the effects of the support system and the overlying soils are confined within the "trough" portion of the settlement; settlements beyond 3H from the excavation are defined purely by the underlying soft clay\(^{130}\).

\(^{130}\) Note: the method described in this chapter is applicable for profiles with underlying normally consolidated clay.
Figure 5.1.1 - Settlement Trough Characteristics for Excavation in Normally Consolidated BBC

Settlement:

$$\delta_v = \alpha \left( e^{ax^2 + bx} \right) \left( 1 + x^2 \right)^c$$

Normalized Settlement:

$$\frac{\delta_v}{\delta_{v(\max)}} = \frac{\left( e^{ax^2 + bx} \right) \left( 1 + x^2 \right)^c}{\beta}$$

Where,

$$\beta = \left( e^{ax_{(\max)}^2 + bx_{(\max)}} \right) \left( 1 + x_{(\max)}^2 \right)^c$$

$$x_{(\max)} = -\frac{b - \sqrt{b^2 - 16ac}}{4a}$$

(location of Maximum Settlement, [m])

and

$$a = 0.00032 + \left[ \frac{1}{(100 - 7.5)(100)} - \frac{1}{(d_B^2 - H)d_B} \right] 0.35$$

$$b = b' + b^*$$

$$c = c' + c^*$$

Values of $a$, $b$, and $c$ can be calculated using charts shown in Figure 5.1.3

Excavation supported by 0.9-m diaphragm wall with rigid struts at $h = 2.5$-m

Figure 5.1.2 - Equation for Describing Settlement Troughs for Undrained Undrained Excavations in Normally Consolidated BBC
\[
\frac{1}{(d_B^* - H)d_B} - \frac{(d_B^* - 3.5H)}{H^2} - \frac{(d_B^*)}{H}
\]

\[
\delta_v = \frac{[e^{a_1 + b_1 + b_2}][1 + x^2][c + c^*]}{\beta}
\]

Note: all dimensions are in meters.

\(d_B^*\) is adj. value based on \(d_b\cdot B\cdot L\) (see Figure 5.1.4)

**Figure 5.1.3 - Calculation of Coefficients for Equation 5.1.2**
I. Wide Excavations: \( B > (d_B - H) \)

\[ d_B^* = d_B \]

II. Intermediate-Width Excavations: \( (d_B - H) \geq B > 2(L-10-H) \)

\[ d_B^* = \frac{(3.5H + B + d_B)}{3} \]

III. Narrow Excavations: \( B \leq 2(L-10-H) \)

\[ d_B^* = \frac{(4.5H + B + L)}{3} \]

Figure 5.1.4 - Definition of Adjusted Depth to Bedrock Parameter \( d_B^* \)
Procedure for Determining the Distribution of Surface Settlements (Excavation Geometry)

Step 1:
Define geometric parameters for $H \geq 7.5$-m
- Excavation Width [$B$]
- Wall Length [$L$]
- Depth to Bedrock [$d_B$]
- Excavation Depth [$H$]
  All dimensions are expressed in [m]

Step 2:
Determine the Adjusted Depth to Bedrock [$d_B^*$]
(Figure 5.1.4)

Step 3:
Determine the values of $a, b^*, b', c^*$, and $c'$
(Figure 5.1.3)

Step 4:
Calculate $\beta$ (Figure 5.1.2)

Step 5:
Calculated the normalized settlement (Figure 5.1.2)

\[ \frac{\delta_v}{\delta_{v(max)}} = f(x) \]
(Figure 5.1.2)

Note: all the dimensions and distances are in meters.
For excavations supported by 0.9-m concrete diaphragm wall
with perfect struts spaced at $h = 2.5$-m.

Figure 5.1.5 - Procedure for Determining the Distribution of Surface Settlements for Excavations in Normally Consolidated Boston Blue Clay
Figure 5.1.6a - Evaluation of Settlement Predictions for Excavations in Normally Consolidated BBC
[B = 80m, NC BBC, DW, H = 15m, h = 2.5m]
Figure 5.1.6b - Evaluation of Settlement Predictions for Excavations in Normally Consolidated BBC
[B = 40m, NC BBC, DW, H = 15m, h = 2.5m]
Figure 5.1.6c - Evaluation of Settlement Prediction for Excavations in Normally Consolidated BBC
[B = 20m, NC BBC, DW, H = 15m, h = 2.5m]
Figure 5.1.7 - Predicted and Actual Maximum Settlements for Undrained Excavations in NC BBC
Prediction of Maximum Settlement, $\delta_{v(\text{max})}$

a. Excavation Width [B] and Excavation Depth [H]

$$\delta_{v(\text{max})}^* = \left( \frac{B}{2m} + 1 \right) n$$

where

- $B$ Excavation Width in meters
- $\delta_{v(\text{max})}^*$ unadjusted Maximum Surface Settlement in cm
- $m = f(L, B)$, see table below
- $n = f(H, B) = i - (H^2 - 7.5^2)j$, see table below

<table>
<thead>
<tr>
<th>B/2 &lt; 15m for all L</th>
<th>B/2 ≥ 15m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>L = 25m</td>
</tr>
<tr>
<td>$m$</td>
<td>7</td>
</tr>
<tr>
<td>$i$</td>
<td>-0.35</td>
</tr>
<tr>
<td>$j$</td>
<td>0.0055</td>
</tr>
</tbody>
</table>

b. Adjustment for Location of Bedrock

$$\delta_{v(\text{max})} = \mu \delta_{v(\text{max})}^*$$

where

- $\delta_{v(\text{max})}$ Maximum Surface Settlement in cm
- $\mu$ Bedrock Adjustment factor, see table below

<table>
<thead>
<tr>
<th>Location of Bedrock</th>
<th>$\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_B \geq 100m$</td>
<td>1.0</td>
</tr>
<tr>
<td>$100m &gt; d_B \geq 75m$</td>
<td>1.05</td>
</tr>
<tr>
<td>$75m &gt; d_B \geq 50m$</td>
<td>1.10</td>
</tr>
<tr>
<td>$50m &gt; d_B \geq 30m$</td>
<td>1.15</td>
</tr>
</tbody>
</table>

Figure 5.1.8 - Prediction of Maximum Surface Settlement for Excavations in NC BBC supported by 0.9-m thick Diaphragm Wall and rigid struts at $h = 2.5m$
Figure 5.1.9a - Illustration of the Predictive Capabilities for Estimating Settlements for Undrained Excavations in NC BBC with $B = 40\text{m}$, $d_B = 50\text{m}$, $L = 25\text{m}$, $h = 2.5\text{m}$ and supported by DW.

**Table:**

<table>
<thead>
<tr>
<th>$H$ [m]</th>
<th>7.5m</th>
<th>10m</th>
<th>12.5m</th>
<th>15m</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a$</td>
<td>0.00015</td>
<td>0.00018</td>
<td>0.00017</td>
<td>0.00016</td>
</tr>
<tr>
<td>$b'$</td>
<td>-0.074</td>
<td>-0.074</td>
<td>-0.074</td>
<td>-0.074</td>
</tr>
<tr>
<td>$b$</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>-0.0037</td>
</tr>
<tr>
<td>$c'$</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>$c^*$</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0175</td>
</tr>
<tr>
<td>$\beta$</td>
<td>5.13</td>
<td>5.17</td>
<td>5.15</td>
<td>5.35</td>
</tr>
<tr>
<td>$d_B^*$</td>
<td>41.25</td>
<td>50</td>
<td>50</td>
<td>50</td>
</tr>
</tbody>
</table>

$m$  | 27 |
$i$  | -0.7 |
$j$  | 0.0115 |
$\mu$| 1.1 |
Figure 5.1.9b - Illustration of the Predictive Capabilities for Estimating Settlements for Undrained Excavations in NC BBC with B = 40m, d_B = 50m, L = 40m, h = 2.5m and supported by DW.
For OCR \leq 1.4

\[ \delta_{v(\text{max})}^* = \left(1 + \frac{B}{2m}\left(\frac{i}{OCR} - \left(H^2 - 7.5^2\right)\frac{j}{OCR^5}\right) \right) \]

For OCR > 1.4

\[ \delta_{v(\text{max})}^* = \left(1 + \frac{B}{2m}\left(\frac{i}{1.4} - \left(H^2 - 7.5^2\right)\frac{j}{1.4^5}\right) \right) \]

Figure 5.2.1 - Estimation of Maximum Surface Settlements for Excavations in Overconsolidated BBC

Page 661
For \([d_s - (H + 2.5)] \leq -2.5\)
\[\lambda = 1.1\]

For \([d_s - (H + 2.5)] > -2.5\)
\[\lambda = 1.1 - 0.04(d_s - H)\]

where \(d_s\) is the depth to top of clay stratum in meters,
\(H\) is the Excavation Depth in meters, and
\(\lambda\) is \((\text{max. sett. with Cohesionless layer of depth } d_s)\) \(\text{ (max. sett. for all clay profile)}\)

Figure 5.2.2 - Adjustment Factor for Overlying Cohesionless Soil, \(\lambda\)
1. Estimate Equivalent OCR = \( \frac{\text{avg. over zone A} + \text{OCR at the toe of the wall}}{2} \)

2. Calculate \( \delta^{*}_{v(\text{max})} \) (see Figure 5.2.1)

3. Determine \( \mu \) and \( \lambda \) (see Figures 5.1.8 & 5.2.2)

4. Calculate \( \delta_{v(\text{max})} = \mu\lambda \delta^{*}_{v(\text{max})} \)

Figure 5.2.3 - Recommendation for Estimating Maximum Surface Settlements, in Composite Soil Profiles Supported
Determining Settlement Trough for Undrained Excavations in Composite Profile consisting of NC BBC supported by Diaphragm Wall and $h = 2.5m$

A. Determine the settlement trough for excavation of the same dimensions in NC BBC (see Figures 5.1.5 and 5.1.8)

B. Calculate the maximum surface settlement for Excavations in the given soil profile (see Figure 5.2.3)

C. Use results in Step B to truncate profile computed in Step A

Figure 5.2.4 - Recommended Method for Estimating the Distribution of Surface Settlement for Excavations in Composite Profile (DW)
Figure 5.2.5a - Illustration of Predictive Capabilities for Composite Soil Profile F1

Excavation Width = B = 40m
Wall Length = L = 40m
Depth to Bedrock = d_b = 50m

Soil Profile: Profile F1

Supported by 0.9-m thick Concrete DW with Rigid Struts spaced at h = 2.5m

Undrained Plane Strain Excavation
Excavation Width = B = 40m
Wall Length = L = 40m
Depth to Bedrock = d_b = 50m
Soil Profile: Profile F3

Supported by 0.9-m thick Concrete DW with Rigid Struts spaced at h = 2.5m
Undrained Plane Strain Excavation

Surface Settlement (in)

Surface Settlement (cm)

Distance from Wall (m)

Figure 5.2.5b - Illustration of Predictive Capabilities for Composite Soil Profile F3

Page 666
Figure 5.2.5c - Illustration of the Predictive Capabilities for Composite Soil Profile F4

Excavation Width = B = 40m
Wall Length = L = 40m
Depth to Bedrock = d_b = 50m
Soil Profile: Profile F4

Supported by 0.9-m thick Concrete DW with Rigid Struts spaced at h = 2.5m

Undrained Plane Strain Excavation
Prediction:

\[
\frac{\text{Max. Surf. Settlement}}{\text{Max. Surf. Sett. (DW w/ rigid supports)}} = \frac{\delta_{v(\text{max})}}{\delta_{v(\text{max})}^{\text{DWw/RS}}} = (1 - \pi) \left[ \frac{\log \left( \frac{k}{300} \right)}{\log (EI_{DW})} \right] + \pi
\]

for NC BBC, \( \pi = 3.5 \)

where \( EI \) is the bending stiffness of the wall in MN-m²/m
\( k \) is the bracing stiffness in MN/m/m

\textbf{Note: For rigid supports, the stiffness is assumed as 300 MN/m/m.}

\textit{Bending Stiffness for Diaphragm Wall, }\( EI_{DW} \text{ is 1442 MN-m²/m}.\)

\textbf{Figure 5.3.1 - Effects of Support Stiffness on Maximum Surface Settlements for Excavations in NC BBC}
Figure 5.3.2a - Illustration of Proposed Support System Stiffness Adjustments for Excavations in Profiles F1 and F2
Figure 5.3.2b - Illustration of Proposed Support System Stiffness Adjustments for Excavations in Profiles F3 and F4
Procedure:

a. Determine soil type and/or OCR for soil between 2.5-m above the excavation grade and 7.5-m below the excavation grade (Zone B)

b. For cohesionless materials, use
\[ \pi = 2.5 \]

Within BBC Layer, (see design chart)
\[ \pi = 3.5 \quad \text{for } OCR = 1.0 \]
\[ \pi = 3.5 - \log(OCR) \left( \frac{3.5 - 1.3}{\log(1.4)} \right) \quad \text{for } 1 < OCR < 1.4 \]
\[ \pi = 1.3 \quad \text{for } OCR \geq 1.4 \]

c. Use the average \( \pi \) over zone B to determine the following ratio

\[ \omega = \frac{\delta_{v(\text{max})}}{\delta_{v(\text{max})}^{\text{DWw/RS}}} = (1 - \pi) \left[ \frac{\log \left( \frac{k}{300} \frac{EI}{EI_{DW}} \right)}{\log(EI_{DW})} \right] + \pi \]

Figure 5.3* - Calculation of Support System Stiffness Adjustment Factor, \( \pi \)
Procedure:

a. Determine settlement trough for excavations of the same geometry but in NC BBC (see Figure 5.1.9a).

b. Calculate $\delta_{\text{v(max)}}$ taking into account of soil profile ($\lambda$, Figure 5.2.4), $d$ ($\varphi$, Figure 5.1.8), and support system stiffness ($\omega$, Figure 5.3.3).

c. Connecting the two curves from steps a and b: for NC BBC Profile Point 1 is on curve b located directly below the maximum point of curve a

Point 2 is on curve a at 3*H from the excavation.

Figure 5.3.4 - Recommended Method for Predicting Surface Settlements with Consideration for Support System Stiffness Effects in NC BBC
Procedure:

a. Determine settlement trough for excavations of the same geometry but in NC BBC (see Figure 5.1.9a).

b. Calculate $\delta_{v(\text{max})}$ taking into account of soil profile ($\lambda$, Figure 5.2.4) $d_b$ ($\mu$, Figure 5.1.8), and support system stiffness ($\omega$, Figure 5.3.3).

c. The trough for composite profile is defined by curve b close to the excavation; beyond point 1, where curves a and b intersects, the settlement is defined by curve a.

Figure 5.3.5 - Recommended Method for Predicting Surface Settlements with Consideration for Support System Stiffness Effects in Composite Profiles
CHAPTER 6

Summary, Conclusions, and Recommendations

The primary goal of this research is to develop improved methods of predicting ground deformations caused by excavation in deep deposits of soft clay. These design recommendations are formulated using non-linear finite element analyses which incorporate the MIT-E3 soil model.

Chapter 2 reviews the existing methods of predicting ground deformations by empirical and numerical methods. This chapter also summarizes the numerical techniques used and developed during the course of this research. The expanded capabilities include (1) implementation of special high order (15-3) triangular finite elements for modeled undrained axisymmetric excavations; and (2) procedures for representing wall and bracing systems.

Chapter 3 presents an extensive parametric study which investigates how excavation geometry, soil profile, and the support system affect the excavation performance. The study focuses on plane strain, undrained excavations in clay dominated soil profiles (with input parameters correspond to Boston Blue Clay). The analyses show that the depth to bedrock is the most significant parameter controlling the shape of the observed
settlement trough. Increases in the preconsolidation pressure and support stiffness generally reduce the magnitude of the deformations, but have limited influence on the deep seated movements and soil displacements far from the excavation.

Chapter 4 includes detailed real-time simulations of three excavations in the Boston area: (1) cross-lot braced excavations for the South Cove rapid transit station; (2) a section of the I-90 extension supported by anchored walls (CA/T South Boston ISS-4); and (3) the proposed braced excavation for a transit-only corridor. The results of the first two case studies suggest that the current method is capable of generating reliable predictions for excavations supported by relatively rigid concrete diaphragm walls; but analyses consistently underestimate measured deflections of sheetpile walls.

Design recommendations for the prediction of surface settlements are presented in Chapter 5 (based on the empirical interpretation of finite element analyses results described in Chapter 3). The far field settlements ("tail" portion of the trough) are controlled by properties of underlying soft clay deposits, while the near field trough portion is dependent on the overlying soil deposits and support system.

6.1 Development of FEA Capabilities of Analyzing Excavations in Clay

The first part of this research addresses the finite element analyses used in simulating excavations in clay (Chapter 2). This research is a continuation and expansion of previous work on numerical analysis of excavations conducted by Hashash [1992]. The prior research showed that more realistic and accurate predictions of deformation patterns and development of failure mechanisms can be captured using non-linear finite element analyses which incorporate the MIT-E3 soil model. Since deformations and stability are two
critical issues in the design and the performance of excavations in soft clay, this non-linear finite element analysis with the MIT-E3 soil model is a promising tool in the evaluating the behavior of excavations in clay. A brief summary of Hashash's work is presented in Section 2.3.

In addition to upgrading this original finite element subroutines (Section 2.4.1), additional numerical techniques were also developed during the course of this research. The expanded capabilities focus on to areas: (1) modeling of axisymmetric undrained excavations (Section 2.4.2), and (2) modeling of excavations supported by tiebacks (Sections 2.4.3).

For typical excavations with limited excavation lengths, plane strain finite element analyses generally overestimate the measured far field ground movements, where kinematic constraints more closely resemble and axisymmetric problem. Undrained analyses impose an important constraint in the simulation of axisymmetric problems. One method of solving this class of problems is to introduce special high order finite elements [after Sloan and Randolph, 1982]. Section 2.4.2 provides detailed descriptions of the implementation of 15-3 triangular elements and the simulation of the excavation process using these elements. Preliminary results showing reductions in the far distance settlements due to axisymmetric assumption are shown in Figure 2.4.2.

The second area of numerical technique focuses on the modeling of structural supports which includes the support wall and the wall supports (see Section 2.4.3). Different support walls are modeled based on their wall bending and axial stiffness. Equivalent elastic solid elements with the same bending and axial stiffness are used to simulate the behavior of different support walls. The different wall supports are modeled using spring elements. Section 2.4.3.3 focuses on the modeling of tiebacks in clay which
includes four spring elements in series: one representing the free length and three representing the fixed length. Numerical examples are included to demonstrate how tieback performance is related to the prestress loading (see Figures 2.4.12 to 2.4.15).

6.2 Parametric Study

Chapter 3 presents eight groups of numerical experiments to examine the effects of excavation geometry, soil profile, and support system. Each numerical experiment consists of plane strain, undrained excavations in a clay-dominate profile with vertical support spacing of \( h = 2.5 \text{-m} \). The parameters covered by the eight groups are summarized below:

<table>
<thead>
<tr>
<th>GROUP</th>
<th>MAIN PARAMETER</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Wall Length (25m ( \leq L \leq 40 \text{m} ))</td>
</tr>
<tr>
<td>B</td>
<td>Excavation Width (15m ( \leq B \leq 80 \text{m} ))</td>
</tr>
<tr>
<td>C</td>
<td>Depth to Bedrock (1.2L ( \leq d_b \leq 100 \text{m} ))</td>
</tr>
<tr>
<td>D</td>
<td>Clay OCR Profile (OCR = 1.0, 1.15, 1.25, 1.7, and 2.0)</td>
</tr>
<tr>
<td>E</td>
<td>Overlying Cohesionless Soil (( d_s = 5 \text{-m and 15-m} ))</td>
</tr>
<tr>
<td>F</td>
<td>Clay Crust (Profiles F1, F2, F3, and F4)</td>
</tr>
<tr>
<td>G</td>
<td>Wall Stiffness (4.3MN-m(^2)/m ( \leq EI \leq 1442 \text{MN-m}^2/\text{m} ))</td>
</tr>
<tr>
<td>H</td>
<td>Bracing Stiffness (37.2 MN/m(^2) ( \leq k_s \leq \text{perfect strut} ))</td>
</tr>
</tbody>
</table>

The results of these eight numerical experiments on movements can be summarized as the following:

1. Wall deflection undergoes three phases of deformation: i) unsupported cantilever deflections; ii) bulging (subgrade bending); and iii) toe kickout. The actual deformation phase is determined by the wall embedment depth (L - H) (Section 3.2.2).
2. As long as the wall deformation remains in the bulging shape, the location of the maximum surface settlement migrates away from the excavation as the excavation progresses. Once wall deformation exhibit toe kickout, the magnitude of the surface settlement continue to increase, but the location of the maximum settlement remains stationary (Section 3.2.2).

3. Deformations increase linearly with increasing excavation width with negligible impact of the stability and distribution of surface settlement. However, for narrow excavations, the location of the trough portion of the settlement tend to be closer to the excavation. (Section 3.2.3)

4. The depth to bedrock has a significant impact on the surface settlement at a distance from the excavation. As the thickness of the clay deposit decreases, the tail portion of the settlement also decrease more rapidly (Section 3.2.4).

5. Changes in the clay stress history profile have tremendous impact on the magnitude as well as the distribution of the surface settlement. As OCR increases, the trough is located further from the excavation with a significant decrease in the magnitude of the settlement (Section 3.3.1).

6. The effect of the overlying cohesionless layer depends on the location of the excavation grade relative to the cohesionless soil / clay interface. If the excavation grade is above the interface, a reduction in the settlement is observed; if the excavation grade is below the interface, a 10% increase in the settlement is observed. However, this effect is restricted within the trough portion of the settlement and has negligible impact on the tail portion of the trough (Section 3.3.2).

7. The presence of a clay crust has negligible impact on the tail portion of the trough, but generally decreases the movements and alters the wall deformation phase (compared to all normally consolidated BBC profile) (Section 3.3.3).

8. Decreasing the wall stiffness causes increases in the wall deflections and soil movements. However, the effects are generally restricted within a limited zone: near the excavation grade for the wall deflection and the trough portion of the settlement curve (Section 3.4.1).
9. The effects of decreasing bracing stiffness are also restricted within a limited zone and does not affect deep seated movements or settlements at the tail portion of the curve. The increase in the wall deflection are generally observed above the excavation grade (Section 3.4.2).

10. Increases in wall and bracing stiffnesses are more effective in reducing movements for soft soils. For excavation in stiffer soils, changes in the support stiffness have a smaller impact (Section 3.4.1 and 3.4.2).

6.3 Case Studies

Chapter 4 presents results from three case studies: (1) South Cove (cross-lot braced excavation); (2) South Boston (wide excavation with walls supported by prestressed tieback anchors); and (3) MBTA Transitway (proposed cross-lot braced excavation). All three case studies involve real-time simulation of the construction activities at each site. The capabilities and limitations of this numerical technique are evaluated based on the comparisons with actual field measurements.

Section 4.1 includes the description of the South Cove site and the evaluation of the analysis results. The South Cove study consists of the evaluation of two sections: Section A-A supported by diaphragm walls with three levels of prestressed cross-lot bracing; and Section B-B supported by sheetpiles with three levels of passive (non-prestressed) struts. Reliable predictions of lateral wall deflections, surface settlements, and pore pressure have been achieved for Section A-A. However, the analyses at Section B-B have not achieved satisfactory agreement with measured lateral wall deflections. The numerical analyses underestimate the wall deflections at this section. Possible causes for this difference include: (1) inaccurate field data since the inclinometer reported zero deflection at the toe of the wall which is embedded in the clay layer; (2) gap between the strut and the wall
since the struts were not preloaded at the sheetpile section; (3) lateral
movements in the wall along the excavation alignment which would cause
reductions in the bending stiffness of the wall; and (4) uncertainties in the soil
behavior of the upper stiff yellow clay.

The South Boston case study is presented in Section 4.2. This case
history refers to the construction of the I-90 extension in Boston,
Massachusetts. The numerical analyses consist of the evaluation of one
section of the roadway, ISS-4. The complete description of this site and
documentation of the construction sequence and field measurements are
included in Whelan [1995]. The ISS-4 section is supported by a concrete
diaphragm wall keyed into the bedrock with three tiers of rock anchors on the
north side; while on the south side, the excavate is supported by a shallower
sheetpile wall with three levels of tieback anchors in the clay crust. Two real-
time analyses were performed at this section: Base Case and Revised Analysis.
The Base Case includes the effect of pumping from the lower aquifer during
the construction period. Based on the assessment of the Base Case results, the
Revised Analysis was performed with the following three modifications: 1)
modification of the base of the mesh to include the underlying 3-m thick
glacial deposits and 6.7-m of the underlying bedrock; 2) refinement of the
permeability properties within the BBC stratum to account for higher
permeability within the clay crust; and 3) omission of the tier 2 tiebacks on the
South wall (2-S tiebacks).

Both analyses achieved very good predictions of the North diaphragm
wall deformations and the far distance (x > 40-m) settlements, but greatly
underestimate the movements at the South sheetpile wall and the
settlements close to the excavation. The difference in the sheetpile
movements can be attributed to the uncertainties in the strength properties of
the surrounding soil, deviations from the assumed plane strain conditions, tieback properties, and installation effects of the sheetpile wall. Possible causes for the large settlements close to the excavation, especially on the North side, are disturbance caused by tieback installation and subsequent influence on the consolidation of the soil. Unfortunately, there were no piezometers placed within the clay outside of the excavation which would allow some assessment of the installation effects of sheetpile wall and the tieback anchors.

Section 4.3 presents results from the evaluation of the proposed braced excavations at the MBTA Transitway site. These analyses were performed as a part of the initial design phase; thus, the limitations and capabilities cannot be evaluated until actual construction has commenced. Four typical sections (Platform, Mezzanine, Transition, and West Tunnel) were evaluated with excavation width ranging 9.8-m to 33.5-m and maximum excavation depths between 17.7-m to 18.6-m. A total of 12 analyses were performed (see Table 4.3.4) for these four sections in order to evaluate the effects of wall length, strut spacing, reduction in undrained shear strength profile, and partial drainage. The results revealed that 3-m reduction in wall length and 1-m increase in strut spacing has negligible effect on the excavation stability and deformation (Table 4.3.5). However, reductions in the undrained shear strength profile can potentially double the maximum deformation at the final invert. Excavations within the Platform section (B = 33.5-m) should be performed in limited lengths; and modifications in the lower level bracing members are needed as predicted loads exceed the design load (see Table 4.3.6).
6.4 Prediction of Surface Settlement

Chapter 5 presents design recommendations for estimating surface settlements under various combinations of excavation geometry, soil profile, and support system. These recommendations incorporate the results of the parametric analyses presented in Chapter 3, therefore, are applicable for profiles with significant underlying soft clay. The recommended procedure includes two components: i) estimation of tail portion of the settlement, and ii) estimation of the maximum settlement, $\delta_{v(\text{max})}$, and the trough.

The recommended procedure uses the underlying soft clay and the excavation geometry (L, B, and $d_B$) to estimate the tail portion of the surface settlement (Figure 5.1.5) which can be described by the following function:

$$\frac{\delta_v}{\delta_{v(\text{max})}} = \left[ e^{(ax^2 + bx)} \right] \left( 1 + x^2 \right)^c \beta$$

The trough portion of the settlement can be obtained by estimating the maximum surface settlement, $\delta_{v(\text{max})}$. This value is estimated using the stress history profile (OCR), excavation geometry (B, L, $d_B$), thickness of overlying cohesionless soil ($d_s$), and support stiffness ($\log [k*EI/300]$).

6.5 Recommendations for Future Work

Though this research has provided a better understanding of how excavation geometry, soil profile, and support system influence the magnitude and distribution of surface settlements for excavations in clay-dominated profile, there are still a number of issues that need to be resolved:

1. In the current analysis procedure, simple elastic-plastic constitutive models are used for soils other than BBC. Proper simulation of the stress-strain-strength behavior for other soil types are also important
especially for excavations supported by sheetpiles. Therefore, additional efforts should focus on incorporating soil models for non-clay materials (such as MIT-S1, Pestana, 1995).

2. The parametric analyses and case studies presented in this thesis assume normalized properties for resedimented BBC. Laboratory tests on undisturbed samples of natural BBC have shown that the strength for natural deposit of BBC is weaker than the strength assumed for resedimented BBC. Consequently, actual displacements are generally bounded by the estimations obtained from two analyses: one matching the measured stress history profile, and the other matching the undrained strength profile (MBTA Transitway analyses). Further refinement of predictions will require revision of MIT-E3 model input parameters for natural BBC.

3. Both the South Cove an South Boston case histories have shown that the current technique tend to underestimate the deformations for excavations supported by sheetpile walls. The difference in the pattern of wall deformations suggest that one possible reason for this deficiency may be due to inappropriate modeling of sheetpile wall as the differences resemble the effects of reduction in the wall stiffness. Lateral movements along the excavation alignment may contribute to this reduction in bending stiffness; hence, additional study on the behavior of sheetpile walls is recommended to determine if lateral buckling is a significant issue in defining the bending behavior of sheetpile walls.

4. All the analyses performed in this thesis assume that the support walls are wished-in-place therefore, causing negligible disturbance. It is well known that sheetpile driving causes large disturbance effects in low permeability clay deposits. Further studies are definitely needed to
investigate if these effects can explain wall movements during excavation.

5. The parametric studies presented in this thesis do not cover a number of key parameters. In particular, further studies are needed to assess the impacts of various prestress schemes, excavation sequencing, and construction duration in controlling ground movements.

6. The prediction method in Chapter 5 focuses only on surface settlements. However, damage to adjacent structures is also affected by lateral movements on the soil. Further study should include these parameters and focus on more detailed modeling of soil-structure interaction.

7. Design methods must also be developed for support structure and prestress schemes.

8. The prediction method for surface settlements presented in Chapter 5 uses the stress history profile as input. Modifications should be made to incorporate the undrained shear strength and modulus of the soil to the design charts.
List of References


Haley and Aldrich, Inc. (1991), "Final geotechnical data report, Central Artery (I-93) / Tunnel (I-90) project, design section D004A, Boston, Massachusetts," submitted to the Massachusetts Department of Public Works, Boston, Massachusetts.

Haley and Aldrich, Inc. (1992), "Draft Report on Pumping Tests No. 3 and No. 4", submitted to the Massachusetts Department of Public Works, Boston, Massachusetts.

Haley and Aldrich, Inc. (1993), "Final report on special laboratory and in situ testing program, Central Artery (I-93) / Tunnel (I-90) project, Boston, Massachusetts," submitted to the Massachusetts Department of Public Works, Boston, Massachusetts.

Haley and Aldrich, Inc. (1993), "Supplemental laboratory testing data, special laboratory and in situ testing program, Central Artery (I-93) / Tunnel (I-90) project, Boston, Massachusetts," submitted to the Massachusetts Department of Public Works, Boston, Massachusetts.


ISPC [International Sheet Piling Company] (1990), Steel Sheet Piling: A selection of the Best, Luxembourg, January.


Parsons Brinckerhoff Quade & Douglas, Inc. (1994), "Geotechnical Data Report, Preliminary design phase, MBTA South Boston Pier / Fort Point Channel Underground Transitway, Boston, Massachusetts," submitted to Stone & Webster Civil and Transportation Services Inc.


APPENDIX A - Sample ABAQUS Input Files and User Subroutines

Figure A1 - Comparison of Input Definition for Excess Pore Pressure (AB AQUS V4.9) versus Total Pore Pressure (AB AQUS V5.4) Formulations

Figure A2 - Comparison of Excavation Procedure used in ABAQUS V4.9 versus ABAQUS V5.4

Figure A3 - Modifications in the User Subroutine UMAT and SIGINI (AB AQUS V4.9 versus V5.4)

Figure A4 - Excavation Procedure for using User Elements (UEL)
ABAQUS Version 4.9

*HEADING
EXCAVATION STUDY
[KG, DAY, METER]
MCC MODEL, NC CLAY
* (define mesh)

*SOLID SECTION, ELSET=CLAY, MATERIAL=CLAY
*MATERIAL, NAME = CLAY
*USER MATERIAL, CONSTANTS=6
0.184, 0.034, 1.1009482, 1.05, 5592.841, 0.957
*DEPVAR
3
*PERMEABILITY, TYPE=ISO, SPECIFIC=1000.0
8.64E-5
*DENSITY
842.0
*INITIAL CONDITIONS, TYPE=RATIO
NALL, 0.957, 0.0, 0.957, -120.0
*INITIAL CONDITIONS, TYPE=STRESS, USER
*RESTART, WRITE, FREQUENCY=100
*STEP, INC=1, CYCLE=3
GEOSTATIC INITIAL STRESS STATE
*GEOSTATIC, PTOL = 10.0
1.E-10, 1.E-10, 1.E-10
*DLOAD
EALL, GRAV, 1.0, 0.0, -1.0, 0.0
**EQUIVALENT WATER PRESSURE ON WALL
WLHS, HP1, 60000.0, +0.0, -60.0
WRHS, HP2, 60000.0, +0.0, -60.0
WBOT, P2, 40000.0
**MAKE UP FOR WT @ 2.5-M
WLHS, P1, -2500.0
WRHS, P3, -2500.0
WBOT, P2, -2500.0
**EQUIVALENT CAP. PRES. AT TOP
ETOP, P4, 2500.0
*BOUNDARY
BOT, 1, 2
BOTM, 8, 117500.0
MCL, 1
MRS, 1
*END STEP

Figure A1 - Comparison of Input Definition for Excess Pore Pressure (ABAQUS V4.9) vs. Total Pore Pressure (ABAQUS V5.4) Formulations
ABAQUS VERSION 4.9

**UNSUPPORTED EXCAVATION**

**ELSET, ELSET=UEL1, GENERATE**
1, 401, 100
**NSET, NSET=UN01, GENERATE**
3, 1003, 100
1001, 1003, 1
**ELSET, ELSET=PEL1, GENERATE**
2, 402, 100
**ELSET, ELSET=PWL1, GENERATE**
501, 501, 1
**STEP, INC=5, CYCLE=15, AMP=RAMP**
**SOILS, CONSOLIDATION, PTOL=10.0, UTOL=10000.0**
1.E-10, 1.E-10, 1.E-10, 1.E-10
**MODEL CHANGE, REMOVE**
UEL1
**BOUNDARY, OP=NEW**
BOT, 1, 2
MCL, 1
MRS, 1
**APPLY EQUIVALENT UNLOADING FOR WATER**
**DLOAD**
PEL1, P4, -2500.0
PWL1, HP1, -60000.0, 0., -60.0
**CAPILLARY EFFECT**
PEL1, P4, 2500.0
PWL1, P1, 2500.0
**END STEP**

ABAQUS VERSION 5.4

**UNSUPPORTED EXCAVATION**

**ELSET, ELSET=UEL1, GENERATE**
1, 401, 100
**STEP, INC=5, AMP=RAMP**
**SOILS, CONSOLIDATION, UTOL=100000.0**
1.E-10, 1.E-10, 1.E-10, 1.E-10
**MODEL CHANGE, REMOVE**
UEL1
**BOUNDARY, OP=NEW**
BOT, 1, 2
MCL, 1
MRS, 1
**CONTROLS, PARAMETER=TIME INCREMENTATION**
25, 25, 11, 25, 10, 4, 12, 5, 6, 3
**CONTROLS, PARAMETERS=FIELD, FIELD=DISPLACEMENT**
0.1, 0.1, , 0.13, , 0.00001
**END STEP**

Figure A2 - Comparison of Excavation Procedure used in ABAQUS V4.9 versus ABAQUS V5.4
ABAQUS VERSION 4.9

I. Arguments for UMAT

SUBROUTINE UMAT(STRESS, STATEV, DDSDE, SSE, SPD, SCD,
RPL, DDSDDT, DRPLDE, DRPLDT,
STRAN, DSTRAN, TIME, DTIME, TEMP, DTEMP, PREDEF, DPRED,
CMNAME, NDI, NSHR, NTENS, NSTATV, PROPS, NPROPS, COORDS,
DROT)
C
IMPLICIT REAL*8(A-H, O-Z)

II. Arguments for SIGINI

SUBROUTINE SIGINI(SIGMA, COORDS, NTENS, NCRDS, NOEL)
C
IMPLICIT REAL*8(A-H, O-Z)

ABAQUS VERSION 5.4

I. Arguments for UMAT

SUBROUTINE UMAT(STRESS, STATEV, DDSDE, SSE, SPD, SCD,
RPL, DDSDDT, DRPLDE, DRPLDT,
STRAN, DSTRAN, TIME, DTIME, TEMP, DTEMP, PREDEF, DPRED,
CMNAME, NDI, NSHR, NTENS, NSTATV, PROPS, NPROPS, COORDS,
DROT, PNEWDT, CELNT, DFGRD0, DFGRD1, NOEL, NPT,
LAYER, KSPT, KSTEP, KINC)
C
INCLUDE 'ABA_PARAM.INC'

II. Arguments for SIGINI

SUBROUTINE SIGINI(SIGMA, COORDS, NTENS, NCRDS, NOEL,
NPT, LAYER)
C
INCLUDE 'ABA_PARAM.INC'

III. Subroutine SDVINI

SUBROUTINE SDVINI(STATEV, COORDS, NSTATV, NCRDS,
NOEL, NPT, LAYER, KSPT)
C
INCLUDE 'ABA_PARAM.INC'
C
STATEV(NSTATV) = 0.0
END

Figure A3 - Modifications in the User Subroutine UMAT and SIGINI (ABAQUS V4.9 versus V5.4)
ABAQUS VERSION 4.9

** UNSUPPORTED EXCAVATION **

*ELSET, ELSET=UEL1, GENERATE
  1, 401, 100
*NSET, NSET=UN01, GENERATE
  3, 1003, 100
  1001, 1003, 1
*ELSET, ELSET=PEL1, GENERATE
  2, 402, 100
*ELSET, ELSET=PWL1, GENERATE
  501, 501, 1
*STEP, INC=5, CYCLE=15, AMP=RAMP
*SOILS, CONSOLIDATION,PTOL=10.0,UTOL=100000.0
  1.E-10, 1.E-10, 1.E-10, 1.E-10
*MODEL CHANGE, REMOVE
  UEL1
*BOUNDARY, OP=NEW
  BOT, 1, 2
  MCL, 1
  MRS, 1
**APPLY EQUIVALENT UNLOADING FOR WATER
*DLOAD
  UEL1, U14, 842.0
  PEL1, U5, -2500.0
  PWL1, U2, -60000.0, 0., -60.0
**CAPILLARY EFFECT
  PEL1, P4, 2500.0
  PWL1, P1, 2500.0
*END STEP

ABAQUS VERSION 5.4 (UEL)

** UNSUPPORTED EXCAVATION **

*ELSET, ELSET=UEL1, GENERATE
  1, 401, 100
*STEP, INC=5, AMP=RAMP
*SOILS, CONSOLIDATION,PTOL=10.0,UTOL=100000.0
  1.E-10, 1.E-10, 1.E-10, 1.E-10
*BOUNDARY, OP=NEW
  BOT, 1, 2
  MCL, 1
  MRS, 1
**APPLY EQUIVALENT UNLOADING FOR WATER
*DLOAD
  UEL1, U14, 842.0
  PEL1, U5, -2500.0
  PWL1, U2, -60000.0, 0., -60.0
**CAPILLARY EFFECT
  PEL1, U5, 2500.0
  PWL1, U6, 2500.0
*CONTROLS, PARAMETER=TIME INCREMENTATION
  25, 25, 11, 25, 10, 4, 12, 5, 6, 3
*CONTROLS, PARAMETERS=FIELD, FIELD=DISPLACEMENT
  0.1, 0.1, , , 0.13, , 0.00001
*END STEP

Figure A4 - Excavation Procedure for using User Elements (UEL)