OPTIMIZATION OF GROUND IMPROVEMENTS AND SANDWICH CONSTRUCTION FOR LAND RECLAMATION APPLICATIONS

by

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THE UNIVERSITY OF NEW BRUNSWICK

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ABSTRACT

The use of deep soil mixing technologies has become increasingly common in North America following its widespread development and adoption in Japan and Scandinavia. This research project considered how design of mass stabilized soils and sandwich construction may be optimized to achieve efficient large scale land reclamation that satisfies both stability and serviceability requirements. In this work the effect of several parameters on the global factor of safety for a land reclamation project was determined. The parameters included the thickness of the soft soil layer, as well as the strength, width, and location of a stabilized soil zone. The second phase of this work studied how dredged/excavated soft soils may be re-used for reclamation or site re-grading. The purpose of this work was to provide guidelines for the economic design of future land reclamation projects using deep soil mixing methods.

Keywords: Land reclamation, ground improvement, sandwich construction, mass soil stabilization, stability, serviceability
DEDICATION

To Mom, who once told me (among many other important things) that “smart people don’t get bored.”
ACKNOWLEDGEMENTS

I would like to acknowledge the contribution of Dr. Arun Valsangkar and Dr. Hany El Naggar. Their mentorship has taught me many lessons which will be carried forward both professionally and personally.

To my colleagues at Stantec Consulting Ltd.: many thanks for your support, time, and resources, which allowed me to pursue this goal.

Lastly, to my friends and family, your kind words and encouragement mean more than I can express.
Table of Contents

ABSTRACT ........................................................................................................................ ii
DEDICATION .................................................................................................................... iii
ACKNOWLEDGEMENTS ............................................................................................... iv
Table of Contents ........................................................................................................... v
List of Tables ................................................................................................................... vii
List of Figures ............................................................................................................... viii
List of Symbols, Nomenclature or Abbreviations ........................................................... x
1.0 Introduction ............................................................................................................. 1
2.0 Optimization of Ground Improvements for Stabilizing Engineered Slopes and MSE Walls Used for Land Reclamation Applications ........................................... 10
   2.1 Introduction .......................................................................................................... 10
   2.2 Related Research ................................................................................................. 11
   2.3 Problem Setup ...................................................................................................... 12
   2.4 Results and Discussion ....................................................................................... 18
      2.4.1 Strength of Ground Improvements ............................................................... 19
      2.4.2 Impact of Slope Angle ................................................................................. 23
      2.4.3 Impact of Width of Ground Improvements ................................................. 25
      2.4.4 Impact of the Location of Ground Improvements ...................................... 28
   2.5 Conclusions ......................................................................................................... 31
   2.6 References .......................................................................................................... 33
3.0 Optimization of Sandwich Construction Using Re-claimed Dredging Waste as Backfill for Land Reclamation Applications ......................................................... 35
   3.1 Introduction .......................................................................................................... 35
   3.2 Related Research ................................................................................................. 36
   3.3 Problem Setup ...................................................................................................... 40
   3.4 Results and Discussion ....................................................................................... 44
      3.4.1 Optimizing the Width of Ground Improvements ........................................ 44
      3.4.2 Impact of Reclaimed Clay on Total Settlement ........................................... 46
   3.5 Conclusions ......................................................................................................... 50
   3.6 References .......................................................................................................... 52
4.0 Summary and Conclusions .................................................................................. 53
5.0 Recommendations for Future Research ............................................................... 56
List of Tables

Table 1-1. Summary of Typical Strengths and Improvement Ratios for Deep Soil Mixing .......... 6
Table 2-1. Equivalent shear strengths based on typical area replacement ratios and soil-cement column strengths .............................................................................................................. 13
Table 2-2. Soil parameters used for slope stability analysis ......................................................... 14
Table 3-1. Summary of cases of sandwich construction used for analysis ................................. 42
Table 3-2. Soil parameters used for slope stability analysis ......................................................... 43
Table 3-3. Soil parameters used for settlement analysis ............................................................... 44
Table 3-4. Results of settlement analysis ..................................................................................... 49
List of Figures

Figure 1-1. Clemson Dam - design of ground improvements (after Wooten et al. 2003) ............... 2
Figure 1-2. Sunset North Basin Dam - design of ground improvements (after Barron et al. 2006) 3
Figure 1-3. Typical application of deep soil mixing for I-95/Route 1 Interchange (from Navin 2005) ................................................................. 4
Figure 1-4. Potential failure mechanisms for deep-mixed columns (after Filz et al. 2012) .......... 7
Figure 2-1. Typical ground improvement applications from the transportation sector; (i) Navin, 2005. and (ii) Archeewa, et. al., 2011 ......................................................... 12
Figure 2-2 Typical sections of the engineered slope problem .......................................................... 14
Figure 2-3. Typical section of MSE Wall problem .......................................................................... 15
Figure 2-4. Variable engineered slope angles ... ............................................................................ 15
Figure 2-5. Variable thickness of soft cohesive soil ..................................................................... 16
Figure 2-6. Typical problem showing range of width of ground improvements ......................... 17
Figure 2-7. Example of varying the location of the ground improvements .................................... 18
Figure 2-8. Relationship between the width of ground improvements and factor of safety for the case of the 3H:1V slope (18°) ................................................................. 19
Figure 2-9. Relationship between the width of ground improvements and factor of safety for the case of the 2.5H:1V slope (22°) ................................................................. 20
Figure 2-10. Relationship between the width of ground improvements and factor of safety for the case of the 2H:1V slope (26°) ................................................................. 20
Figure 2-11. Relationship between the minimum required width of ground improvements to achieve FS = 1.5 and the strength of ground improvements for the case of the 3H:1V slope (18°) .................................................................................. 21
Figure 2-12. Relationship between the minimum required width of ground improvements to achieve FS = 1.5 and the strength of ground improvements for the case of the 2.5H:1V slope (22°) ................................................................. 21
Figure 2-13. Relationship between the minimum required width of ground improvements to achieve FS = 1.5 and the strength of ground improvements for the case of the 2H:1V slope (26°) ................................................................. 22
Figure 2-14. Relationship between width of ground improvements and factor of safety for an MSE wall ................................................................................................. 23
Figure 2-15. Relationship between the minimum required width of ground improvements to achieve FS = 1.5 and the strength of ground improvements for an MSE wall ...... 23
Figure 2-16. Relationship between the factor of safety and width of ground improvements based on the slope of the engineered slope/MSE wall – D = 10m .................................................. 24
Figure 2-17. Relationship between factor of safety and width of ground improvements based on the slope of the engineered slope/MSE wall – D = 20 m .................................................. 24
Figure 2-18. Relationship between factor of safety and width of ground improvements based on the slope of the engineered slope/MSE wall – D = 30 m .................................................. 25
Figure 2-19 (i-iii). Limited effectiveness of increasing B, widths of ground improvements equal to 10 m, 20 m, and 30 m, respectively ................................................................. 27
Figure 2-20. Relationship between the location of ground improvement and factor of safety for the case of the 3H:1V slope (18°) ................................................................. 28
Figure 2-21. Relationship between the location of ground improvement and factor of safety for the case of 2.5H:1V slope (22°) ................................................................................ 29
Figure 2-22. Relationship between the location of ground improvement and factor of safety for the case of the 2H:1V slope (26°) ............................................................................. 29
Figure 2-23. Relationship between the location of ground improvements and factor of safety for MSE walls ................................................................................................................. 31
Figure 3-1. Pneumatic flow mixing apparatus (after Kitazume and Satoh, 2003) ......................... 37
Figure 3-2. Typical cross section of the Changi East Reclamation Project (after Choa 1994) ...... 37
Figure 3-3. Typical cross section of the Port Botany Expansion Project (after Davies and Mellquham, 2011) ................................................................................................................. 39
Figure 3-4. Typical terrestrial application of using excavated soft soils for re-grading ..........40
Figure 3-5. Representative cross section of problem setup...................................................... 41
Figure 3-6. Typical cross section illustrating sandwich construction........................................ 42
Figure 3-7. Optimization of ground improvements for Case 1, 100% granular backfill ............ 45
Figure 3-8. Optimization of ground improvements for Case 8, 80% reclaimed clay and 20% granular backfill ........................................................................................................ 45
Figure 3-9. Optimized widths of ground improvements .......................................................... 46
Figure 3-10. Tradeoff comparison between settlement and widths of ground improvements for sandwich construction ........................................................................................................ 50
### List of Symbols, Nomenclature or Abbreviations

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\delta_{\text{elastic}}$</td>
<td>elastic deformation</td>
</tr>
<tr>
<td>$\Delta \sigma$</td>
<td>change in total stress</td>
</tr>
<tr>
<td>$\sigma_{o}'$</td>
<td>initial effective stress</td>
</tr>
<tr>
<td>$c_a$</td>
<td>coefficient of secondary consolidation</td>
</tr>
<tr>
<td>$c_c$</td>
<td>coefficient of consolidation</td>
</tr>
<tr>
<td>$e_o$</td>
<td>initial void ratio</td>
</tr>
<tr>
<td>CDSM</td>
<td>Cement Deep Soil Mixing</td>
</tr>
<tr>
<td>E</td>
<td>Young’s modulus</td>
</tr>
<tr>
<td>H</td>
<td>Thickness of soil unit</td>
</tr>
<tr>
<td>$H_c$</td>
<td>Thickness of clay</td>
</tr>
<tr>
<td>MSE</td>
<td>Mechanically Stabilized Earth</td>
</tr>
<tr>
<td>$S_p$</td>
<td>Primary consolidation settlement</td>
</tr>
<tr>
<td>$S_{\text{sec}}$</td>
<td>Secondary consolidation settlement</td>
</tr>
<tr>
<td>$S_{\text{total}}$</td>
<td>Total settlement</td>
</tr>
<tr>
<td>$t_p$</td>
<td>time to complete primary consolidation</td>
</tr>
<tr>
<td>$t_{\text{sec}}$</td>
<td>time to complete secondary consolidation</td>
</tr>
</tbody>
</table>
1.0 Introduction

Ground improvement methods such as stone columns, deep soil mixing, vibrocompaction, etc., are becoming increasingly popular in North America, as a means of remediating soft soil sites identified as being susceptible to excessive deformation, liquefaction, or having inadequate bearing capacity for the intended use. Deep soil mixing is an in-situ soil treatment, where soil is mixed with binders that are injected through a hollow auger in either a dry or slurry form. The cemented soil material generally has a higher strength, lower permeability, and lower compressibility than the native soil (Bruce 1999). A site can be stabilized either by forming columns of stabilized soil (column stabilization) or by stabilizing a large mass of soil (mass stabilization) (EuroSoilStab 2002).

The San Pablo Dam, the Clemson Upper and Lower Dams, and the Sunset North Basin Dam projects are all examples of case studies in the United States, where deep soil mixing ground improvements have been implemented in the seismic remediation of potentially liquefiable soils under foundations of dams. In the case of the San Pablo Dam located in El Sobrante, CA, Cement Deep Soil Mixing (CDSM) was used to improve a swath of alluvial/colluvial materials to allow for a downstream buttress to be constructed at the toe of the dam (Kirby et al. 2010). Deep soil mixing was utilized at the Clemson Upper and Lower Division Dams in Pickens County, SC to construct 1 m (3ft) thick, 15.2 m (50 ft) long transverse shear walls oriented perpendicular to the axis of the dam.
and a 3-foot-thick longitudinal wall parallel to the dam axis at the upstream end of the transverse walls. The deep mixed elements were constructed under the existing toe berm to provide reinforcement to resist excessive deformations induced by a seismic event (Wooten et al. 2003). The Sunset North Basin Dam in San Francisco, CA also used CDSM to remediate foundation soils susceptible to liquefaction during a seismic event. In this case, the designers recommended three zones of remediation, each comprised of multiple discrete blocks. The discrete block layout included gaps between the individual blocks in order to provide a conduit for regional groundwater flow (Barron et al. 2006). Figures 1-1 and 1-2, respectively, show the Clemson Dam and Sunset North Basin Dam designs.

Figure 1-1. Clemson Dam - design of ground improvements (after Wooten et al. 2003)
Figure 1-2. Sunset North Basin Dam - design of ground improvements (after Barron et al. 2006)

Deep-mixed column supported embankments are widely used in Japan, Scandinavia, and increasingly, the United States, to mitigate excessive settlements and minimize impact to adjacent existing infrastructure. The I-95/US Route 1 interchange reconstruction project in Alexandria, VA had an extensive deep mixing stabilization component. The project consisted of a 6-lane bridge being replaced with a 12-lane bridge, which also required significant realignment and widening of the approach interchanges. The in-situ soils consisted of soft organic silts and clays approximately 10 m thick overlying silty clay; a 2 m thick layer of sand and gravel separated the two clay units. Use of conventional staged construction and vertical drains was not possible for two reasons, (i) the time required for consolidation did not fit within the project construction schedule and (ii) the
consolidation settlements incurred by the construction of new embankments would have significant impacts on existing embankments and deep foundations located immediately adjacent to the new structure. Soil cement buttresses constructed using overlapping soil-cement columns were constructed at the toe of new embankments to provide acceptable global stability. These buttresses were also used in some locations as a base for Mechanically Stabilized Earth (MSE) retaining walls. Individual soil cement columns were used directly underneath the embankment (Figure 1-3) to support the loads, significantly reducing the total settlements (Lambrechts et al. 2003, Stewart et al. 2004, and Navin 2005).

Figure 1-3. Typical application of deep soil mixing for I-95/Route 1 Interchange (from Navin 2005)

Applications similar to those used at the I-95/Route 1 interchange were developed and are frequently used in the Scandinavian region of Europe, and in Southeast Asia.
Examples of these applications are deep column mixing used to reduce settlements at bridge approaches (Lin and Wong 1999) or embankments (Bergado et al. 1999).

Deep soil mixing has also been used in Louisiana to stabilize levees and floodwalls (Adams 2011 and Filz et al. 2012) and in British Columbia to stabilize an unstable slope affecting transportation infrastructure (Lapointe 2012). In both cases, deep soil mixing was utilized to increase the undrained shear strength of soils near the toe of slopes in order to increase the global factors of safety against circular shear failure.

A summary of typical strengths and improvement ratios used in practice is provided in Table 1-1. The unconfined compressive strength of improved soil varies widely and target strengths and replacement ratios (in the case of columns or panels) are dependent on the specific project requirements. For limit equilibrium analysis it is recommended to assume shear strength equal to 40% to 50% of the unconfined compressive strength for the ground improvements (Wooten and Foreman 2005 and Bruce 2001). It is also recommended by Filz et. al., (2012) that, in the application of limit equilibrium slope stability analysis, the residual strength of deep-mixed soils be used to account for potential for progressive failures.
<table>
<thead>
<tr>
<th>$q_u$ (kPa)</th>
<th>Area Replacement Ratio</th>
<th>Columns/Mass Stabilization</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 - 750</td>
<td>n/a</td>
<td>mass stabilization</td>
<td>Kitazume and Satoh, 2003</td>
</tr>
<tr>
<td>200 – 480</td>
<td>n/a</td>
<td>mass stabilization</td>
<td>Kitazume and Hayano, 2007</td>
</tr>
<tr>
<td>100 - 500</td>
<td>n/a</td>
<td>mass stabilization</td>
<td>EuroSoilStab, 2002</td>
</tr>
<tr>
<td>600</td>
<td>0.13</td>
<td>columns</td>
<td>Bergado, <em>et al.</em>, 1999</td>
</tr>
<tr>
<td>2800$^1$</td>
<td>0.2</td>
<td>columns</td>
<td>Wooten, <em>et al.</em>, 2003</td>
</tr>
<tr>
<td>2000$^1$</td>
<td>0.5</td>
<td>columns</td>
<td>Kirby, <em>et al.</em>, 2010</td>
</tr>
<tr>
<td>380</td>
<td>0.2</td>
<td>columns</td>
<td>Navin and Filz., 2006</td>
</tr>
<tr>
<td>690 - 1100</td>
<td>0.2 – 0.3</td>
<td>columns</td>
<td>Lambrechts, <em>et al.</em>, 2004</td>
</tr>
<tr>
<td>240 – 480</td>
<td>0.06 – 0.2</td>
<td>columns</td>
<td>Stewart, <em>et al.</em> 2004</td>
</tr>
<tr>
<td>700 - 1000</td>
<td>0.16 – 0.8</td>
<td>columns</td>
<td>Shao and Ivanetich, 2010</td>
</tr>
<tr>
<td>700 - 1000</td>
<td>not stated</td>
<td>columns</td>
<td>Dasenbrock, 2005</td>
</tr>
<tr>
<td>500 - 1000</td>
<td>not stated</td>
<td>columns</td>
<td>Terashi, 2004</td>
</tr>
<tr>
<td>200 - 860</td>
<td>0.35 – 0.44</td>
<td>columns</td>
<td>Esrig, <em>et al.</em>, 2004</td>
</tr>
<tr>
<td>200 – 2000$^1$</td>
<td>not stated</td>
<td>columns</td>
<td>Bruce, 2001</td>
</tr>
</tbody>
</table>

$^1$ Higher strength of improved ground generally corresponds to granular in situ soils therefore were not considered because they do not represent strengths that could be realistically achieved for the cohesive soil conditions assumed for this study.

Filz and his students have continued to build up on the work completed by Kivel and Broms (1999) where it was demonstrated that in the case of deep mixed columns there
are multiple modes of failure in addition to the assumed shear failure, ranging from tilting to buckling and bulging, which must be considered. The possible modes of failure are shown in Figure 1-4. In the case of deep mixed columns, it has been demonstrated that limit equilibrium slope stability analysis based on shear failure can be non-conservative (Terashi 2003, and Filz et al. 2012). Therefore it may be useful to represent an area improved with soil-cement columns as a homogeneous mass with equivalent shear strength for determining the global stability of an area; however, the global shear failure may not govern the final detailed design.

Figure 1-4. Potential failure mechanisms for deep-mixed columns (after Filz et al. 2012)

Adams et al. (2010) considered the optimization of deep mixed shear walls for a pile-supported, reinforced, concrete flood wall. The study considered the influence that the width, and location of deep mixed soils relative to the flood wall, had on the global factor of safety. In the case of Lapointe (2012), the focus was on the physical properties of the mixed soils in the laboratory and in the field. It is understood that the physical design of the improvements were completed by a private consultant and has not been published at
this time. In her conclusions Lapointe suggested that additional work is required to study the influence of plasticity of soil on the strength of the cement-treated soil.

In addition to transportation applications there has also been published case studies related to the use of deep soil mixing and jet grouting to support heavy structures such as tanks, silos, and storage buildings (Shao 2009, and Shao and Ivanetich 2010).

Marine shipping facilities are another type of infrastructure, which, based on their inherent locations, are frequently subject to soft soil conditions. Geomorphology of near shore deposits typically results in thick deposits of normally consolidated silts and clays, which are susceptible to excessive settlements and slope instability. As is the case with most infrastructure, constraints due to limited available land, and the fixed location of the proposed users, means relocation to avoid difficult geotechnical conditions is an unacceptable alternative. Often it is uneconomical to construct these facilities due to the cost and environmental implications of dredging soft soils, and building up the site with structural fill.

In the practice of using deep mixed soils to control settlements of embankments constructed on soft soil, the focus of design appears to be on specifying and achieving physical properties of a treated soil, and to a lesser degree, examining the benefits of various column geometries. So far, it appears that the optimization of a design has
overlooked the impact that the location of ground improvements has, in addition to the overall strength and width of the improved zone.

The literature related to the practice of sandwich construction is very limited. Sandwich construction is a method of construction where soft soils typically considered to be a waste product may be used as backfill when placed in alternating lifts of a stronger granular aggregate. When sandwich construction is mentioned, there is no discussion as to what considerations were made to determine the quantities used and any other limiting factors.

The use of deep soil mixing to provide adequate bearing capacity and global stability for large land reclamation projects and marine shipping facilities is an acceptable method of addressing geotechnical problems of soft soils. The purpose of this work is to evaluate the effect of strength, width, and location of mass soil stabilization on the global factor of safety against slope failure. A factor of safety equal to 1.5 is typically accepted for long term stability of slopes and is determined using limit equilibrium analysis. and the second purpose of this work is to evaluate the effect of sandwich construction using re-claimed dredged soil on serviceability for slopes and vertical earth retaining structures founded on soft soils. The goal of this study is to develop some general rules that may be used in developing a conceptual design for similar applications.

The thesis format is in accordance with each chapter being written as a possible publication.
2.0 Optimization of Ground Improvements for Stabilizing Engineered Slopes and MSE Walls Used for Land Reclamation Applications

2.1 Introduction
Ground improvement methods such as stone columns, deep soil mixing, vibrocompaction, etc., are becoming increasingly popular in North America as a means to remediate soft soils susceptible to excessive deformation, liquefaction, or having inadequate bearing capacity for the intended use (Bruce 2012). The San Pablo Dam, Sunset North Basin Dam, and the Clemson Upper and Lower Dams project are all examples of case studies in the United States where ground improvements have been implemented in the seismic remediation of potentially liquefiable soils under foundations of dams and embankments (Barron et al. 2006, Kirby et al. 2010, Mitchell 2008, and Wooten et al. 2003). Column supported embankments are widely used in Japan, Scandinavia, and increasingly the United States to mitigate excessive settlements and minimize impact to adjacent existing infrastructure. Deep soil mixing has also been used in Louisiana, United States and British Columbia, Canada, to stabilize levees and floodwalls, and an unstable slope affecting transportation infrastructure, respectively.

Marine shipping facilities are another type of infrastructure which, based on their inherent locations, are frequently need to be constructed on undesirable soil conditions. Geomorphology of near shore deposits typically result in thick deposits of normally to slightly consolidated silts and clays, which are susceptible to excessive settlements and slope instability. Often it is uneconomical to construct these facilities due to the cost and environmental implications of dredging soft soils and rebuilding the site with structural
fill, especially when the soft soil layer is relatively thick and/or the considered area is large.

The use of deep soil mixing to provide adequate bearing capacity and global stability for large land reclamation projects and marine shipping facilities to be located on soft soils may be an acceptable method of addressing most geotechnical problems. The purpose of this work is to evaluate the effect of strength, width, and location of mass soil stabilization on the global factor of safety against slope failure for slopes and vertical earth retaining structures for land reclamation purposes. The goal of this study is to develop some general guidelines that may be used in developing a conceptual design for similar applications.

2.2 Related Research

Research related to the physical properties of deep mixed soils has been ongoing for decades. It is known that the physical properties of a treated soil are a function of the characteristics of the stabilizing agent and the in-situ soil, the mixing conditions, and the curing conditions (CDIT 2002, EuroSoilStab 2002, Adams 2011, and Navin 2005). Deep soil mixing has been gaining acceptance across North America as a solution for inadequate bearing capacity and excessive settlements under highway or railroad embankments (Navin 2005 and Archeewa et al. 2011), excessive deformations during excavations (O’Rourke and McGinn 2006, and Rutherford et al 2006) and slope instability of both natural and manmade structures (Adams 2011 and Lapointe 2012). Figure 2-1 illustrates examples of how deep mixed columns have been designed for transportation applications.
In the practice of using deep mixed soils to control settlements of embankments constructed on soft soil, the focus of design appears to be on specifying and achieving physical properties of a treated soil, and to a lesser degree, examining the benefits of various column geometries. So far, the optimization of a design has overlooked the impact that the location of the ground improvements have in addition to the overall strength and width of the improved zone.

Figure 2-1. Typical ground improvement applications from the transportation sector; (i) Navin, 2005. and (ii) Archeewa, et. al., 2011

2.3 Problem Setup

A theoretical model was used to evaluate the sensitivity of global factor of safety to several variables affecting the behavior of deep soil mixing stabilization. The model was developed based on common soft soil conditions. The work was based on evaluating the global factor of safety of a retaining structure constructed on a thick layer of soft, cohesive soil overlying bedrock. The undrained shear strength of the clay was assumed to be 25 kPa. Analyses for this study included both engineered slopes and Mechanically
Stabilized Earth (MSE) walls. A limit equilibrium software package was used to run all undrained analyses (SLOPE/W, 2012).

Figure 2-2 illustrates the problem geometry that was utilized to evaluate engineered slopes. For the purpose of this work the height of the slope was maintained at 10 m and the reclaimed land behind the slope was assumed to have a horizontal surface. The ground improvements were assumed to be representative of a mass soil stabilization application (homogeneous) and the composite strength of the stabilized zone ranged between 50 kPa and 400 kPa. The strength was represented as undrained shear strength in the numerical model, and is based on typical strengths used in the cases of mass stabilization that are summarized in Table 1-1. Further, the strengths of a homogeneous mass equivalent to individual columns and in-situ soils based on the typical strengths of soil-cement columns and the respective area replacement ratio were found to be similar to those reported for mass stabilization (see Table 2-1).

Table 2-1. Equivalent shear strengths based on typical area replacement ratios and soil-cement column strengths

<table>
<thead>
<tr>
<th>ARR</th>
<th>0.2</th>
<th>0.3</th>
<th>0.4</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
<th>0.8</th>
</tr>
</thead>
<tbody>
<tr>
<td>(c_{u_{\text{min}}}^{1,2}) (kPa)</td>
<td>58</td>
<td>74</td>
<td>90</td>
<td>107</td>
<td>123</td>
<td>139</td>
<td>156</td>
</tr>
<tr>
<td>(c_{u_{\text{max}}}^{1,2}) (kPa)</td>
<td>114</td>
<td>159</td>
<td>204</td>
<td>248</td>
<td>293</td>
<td>337</td>
<td>382</td>
</tr>
</tbody>
</table>

1Shear strength determined assuming that \(c_u = 50\%\) UCS (Bruce 2001).
2Shear strength of the un-improved soils assumed to be 25 kPa.

The soil parameters used for the various materials during the slope stability analyses are summarized in Table 2-2.
During the parametric analyses, each of the following input parameters were varied: slope angle, thickness of cohesive soil, width of the ground improvements, strength of the ground improvements, and the location of ground improvements with respect to the toe of the slope.

![Figure 2-2 Typical sections of the engineered slope problem](image)

Table 2-2. Soil parameters used for slope stability analysis

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit Weight (kN/m$^3$)</th>
<th>$\Phi^*$ (°)</th>
<th>$c_u$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>base sand</td>
<td>22.0</td>
<td>40</td>
<td>-</td>
</tr>
<tr>
<td>in-situ clay</td>
<td>15.0</td>
<td>-</td>
<td>25</td>
</tr>
<tr>
<td>fill sand</td>
<td>18.0</td>
<td>30</td>
<td>-</td>
</tr>
<tr>
<td>embankment</td>
<td>19.6</td>
<td>35</td>
<td>-</td>
</tr>
<tr>
<td>ground improvements</td>
<td>15.0$^1$</td>
<td>-</td>
<td>100 to 400</td>
</tr>
</tbody>
</table>

$^1$The characteristic unit weight of stabilized soil columns is equal to that of unstabilized soil columns (EuroSoilStab 2002).

In the case of applications where areal extent and right of way are limited, or perhaps a marine terminal where a sloping surface would prohibit access by ships, the problem was
also considered using a vertical MSE wall as shown in Figure 2-3. Only the global stability of the MSE was evaluated in this study. The internal stability was assumed to be acceptable for the purpose of this analysis.

Figure 2-3. Typical section of MSE Wall problem

Three slope angles (α) were evaluated as shown in Figure 2-4. The slope angles 18°, 22°, and 26° correspond to slopes frequently used in practice, 3H:1V, 2.5H:1V, and 2H:1V, respectively.

Figure 2-4. Variable engineered slope angles
The thickness (D) of the soft, cohesive layer was varied to range between 10 m, 20 m, and 30 m as shown in Figure 2-5. The width of the ground improvements zone (B) were also varied between 5 m and 40 m, in 5 m increments, as shown in Figure 2-6. To minimize duplication of similar figures only the MSE wall cross section is used here to illustrate the variation in thicknesses of the stabilized zone, however, both engineered slopes and MSE walls were evaluated in all cases.

Figure 2-5. Variable thickness of soft cohesive soil
In addition to examining the sensitivity of factor of safety relative to the variables described previously, the effect of the locations of the ground improvements was also evaluated. This aspect was studied by translating the ground improvement zone either inward or outward relative to the toe of the wall/slope. The magnitude of the shift was measured as a percentage of the total width of ground improvements in order to determine if the behavior was similar or dissimilar for all widths of ground improvements. The zone of ground improvements were moved in increments of 25%\(B\).
The neutral position of ground improvements are considered to be when the outward limit of the ground improvement coincides with the toe of the wall/slope, as shown in Figure 2-6. Shifts in the outward and inward directions are represented by negative and positive sign conventions, respectively, and are measured from the outward limit of the ground improvements. Figure 2-7 illustrates a +50%B shift, or in the case of a 20 m wide zone it has been shifted 10 m in the outward direction.

Figure 2-7. Example of varying the location of the ground improvements

2.4 Results and Discussion

The results of the extensive parametric study that was conducted to investigate the effect of strength, width, and location of mass soil stabilization on the global factor of safety against slope failure for slopes and vertical earth retaining structures are presented in this section. Within the following discussion and supporting figures note that, unless stated
otherwise, the ground improvements are located at the neutral location relative to the wall/slope.

2.4.1 Strength of Ground Improvements
It is inherently understood that as the composite strength of the ground improvements increases, the factor of safety would also increase. Figures 2-8 to 2-10 present results from analysis completed for a slope constructed on a 20 m thick clay deposit. The results are similar to those with 10 m and 30 m thick soft soil deposits. The relationship between the strength and factor of safety is generally linear; however, the gradient is a function of the strength of the improved ground. As the strength of improved ground increases, the effect of increasing the width of the improved ground (B) results in increased factor of safety.

Figure 2-8. Relationship between the width of ground improvements and factor of safety for the case of the 3H:1V slope (18°)
Figure 2-9. Relationship between the width of ground improvements and factor of safety for the case of the 2.5H:1V slope (22°)

Figure 2-10. Relationship between the width of ground improvements and factor of safety for the case of the 2H:1V slope (26°)

Using the results of the cases summarized in Figures 2-8 to 2-10, the minimum required width of ground improvement to achieve a factor of safety of 1.5, was determined. The results are shown in Figure 2-11 to Figure 2-13. There is an almost bi-linear relationship between the strength of ground improvements and the minimum required width. The
results clearly indicate that after the strength exceeds 200 kPa the benefit of increasing the strength, in terms of the factor of safety, is limited. In all cases the minimum required strength is 100 kPa.

Figure 2-11. Relationship between the minimum required width of ground improvements to achieve FS = 1.5 and the strength of ground improvements for the case of the 3H:1V slope (18°)

Figure 2-12. Relationship between the minimum required width of ground improvements to achieve FS = 1.5 and the strength of ground improvements for the case of the 2.5H:1V slope (22°)
Figure 2-13. Relationship between the minimum required width of ground improvements to achieve FS = 1.5 and the strength of ground improvements for the case of the 2H:1V slope (26°)

In the case of a MSE wall, or $\alpha = 90^\circ$, the relationship between the width of ground improvements and the factor of safety is also linear. Similar to the case of engineered slopes the gradient of the relationship is also a function of the strength although it is observed that generally the effect of increasing the width is not as significant for MSE structures as it is for slopes. This result is shown in Figure 2-14. The result of comparing the minimum width of ground improvement required to achieve a factor of safety of 1.5 varies in the case of an MSE wall more than it does for an engineered slope, as shown in Figure 2-15. The minimum required width increases as the thickness of the soft, cohesive deposit increases. For soft soil thicknesses up to 20 m the minimum required strength is 150 kPa and in the case of a 30 m thick deposit the minimum required strength increases to 200 kPa.
Figure 2-14. Relationship between width of ground improvements and factor of safety for an MSE wall

Figure 2-15. Relationship between the minimum required width of ground improvements to achieve FS = 1.5 and the strength of ground improvements for an MSE wall

2.4.2 Impact of Slope Angle

The analysis was also used to determine if there is a relationship between the slope angle and the factor of safety under consistent conditions (soft soil thickness and strength of ground improvements). Figures 2-16 to 2-18 present typical results for soft soil thicknesses of 10 m, 20 m, and 30 m, respectively, for improved ground strength of
250 kPa. In the case of engineered slopes there is a linear relationship between the width of ground improvements and factor of safety, however, the gradient of the relationship is not dependent on the angle of the slope (\(\alpha\)) rather it is dependent on the thickness of the soft strata.

Figure 2-16. Relationship between the factor of safety and width of ground improvements based on the slope of the engineered slope/MSE wall – \(D = 10\) m

Figure 2-17. Relationship between factor of safety and width of ground improvements based on the slope of the engineered slope/MSE wall – \(D = 20\) m
In the case of the MSE wall ($\alpha = 90^\circ$) the relationship is bilinear. In the cases of 10 m and 20 m thick soft soil layers the factor of safety is dependent on the width of the ground improvement only up to a point after which the factor of safety is unchanged with increased width of ground improvements. This change occurs when $B$ is approximately equal to $D$ and is explained in further detail under Section 2.4.3.

2.4.3 Impact of Width of Ground Improvements

The impact of the width of ground improvements to factor of safety of engineered slopes is illustrated in Figures 2-8 to 2-10, 2-14, and 2-16 to 2-18. Linear relationships exist that are dependent on the width and strength of the ground improvements, and the thickness of the soft strata. The aforementioned figures present the general relationships. An effort was made to quantitatively define the relationships, however it was determined that fourth order polynomial function would be required and still would not produce a simple solution.
As indicated previously in Section 2.4.2, in the case of MSE walls the relationship between the width of ground improvements and the factor of safety for soft soils 10 m and 20 m thick is bilinear (Figure 2-16 and Figure 2-17). In the case of \( D = 30 \) m the relationship is linear and consistent with the behavior of the engineered slopes (Figure 2-18). The bilinear relationship for \( D = 10 \) m and \( D = 20 \) m exists because as the width of ground improvements increase the failure circles are forced upward, becoming shallower. For any given strength of improved ground, there is a point at which the slip circle becomes shallow enough that it does not pass through the entire width of the improved ground. Therefore, additional ground improvements beyond the slip surface are not effective in increasing the factor of safety. This is illustrated below in Figure 2-19i through 2-19iii. It is noted that for soft soil thicknesses of 30 m the behavior is similar to that of engineered slopes because the soils are thick enough to permit a failure deep enough to encompass the entire widths of ground improvements being evaluated in this study.
Figure 2-19 (i-iii). Limited effectiveness of increasing B, widths of ground improvements equal to 10 m, 20 m, and 30 m, respectively.
2.4.4 Impact of the Location of Ground Improvements

It was anticipated at the start of this work that the relationship between the location of ground improvements and factor of safety would be parabolic. The result of varying the locations of the ground improvements relative to the toe of the engineered slope indicates that there is a finite area within which the ground improvements are most effective. As shown in Figures 2-20 to 2-22 this range lies between 25%B inward to 50%B outward where B is the total width of ground improvements and inward/outward are with respect to the toe of the slope. The effect of changes in location of ground improvements are more pronounced as the width of the ground improvements increase. This is attributed to the fact that for larger widths, the magnitude of the translation of ground improvements with respect to the toe is more significant compared to the overall footprint of the slope i.e. for a constant footprint of 20 m to 30 m (footprint of a 2H:1V or 3H:1V, 10 m high slope, respectively) shifting 25% of 5 m (1.25 m) in either direction is expected to have less of an impact on the factor of safety than 25% of 20 m (5 m).

Figure 2-20. Relationship between the location of ground improvement and factor of safety for the case of the 3H:1V slope (18°)
The results also indicate that it is more effective to provide ground improvements outward relative to the toe than inward. However, the limits of the range of effectiveness on the inward and outward sides are dependent on different factors. Translating the ground improvement a relatively short distance ($\geq 25\%B$) in the inward direction causes the toe of the engineered slope to be completely unsupported, thus unacceptable factors.
of safety are the result in a localized but critical location. Global factor of safety is more
tolerant to shifts in the outward direction because the ground improvements contribute to
a significant portion of the failure surface for a larger distance; up to 50%B. Once the
ground improvements are located >50%B in the outward direction the contribution of
shear strength of the improved zone to the failure surface is insufficient and the factor of
safety is decreased.

The result of varying the locations of the ground improvements relative to the toe of MSE
walls indicates that there is a finite area within which the ground improvements are
effective once the width of ground improvements exceed 25 m. The effect on factor of
safety by varying the locations of ground improvements less than 12 m is negligible, and
is very limited for ground improvements 15 to 20 m wide as shown in Figure 2-23. This
is for the same reason described for engineered slopes that for smaller widths, the
translation of ground improvement does not have a significant impact on the shear
strength contribution to the factor of safety. For ground improvements with a minimum
of 25 m width the most effective location is shifted 25%B to 75%B outward in relation to
the toe of the MSE wall. Similar to engineered slopes a relatively small shift towards the
inward direction results in the footprint of the MSE wall being unsupported causing
insufficient factors of safety, and a shift in the outward direction increases the
contribution to shear strength along the failure plane within the range noted above.
2.5 Conclusions
Based on the results of the extensive parametric study that was conducted in this study to investigate the effect of strength, width, and location of mass soil stabilization on the global factor of safety against failure for slopes and vertical earth retaining structures, the following conclusions have been drawn regarding the design of ground improvements for global stability of engineered slopes and MSE walls:

- The minimum required strength of ground improvements was 100 kPa.
- The efficiency of the strength of ground improvements becomes asymptotic after strength exceeds 200 kPa.
- For engineered slopes, the slope angle has negligible impact on the overall factor of safety.
- For engineered slopes, there is a linear relationship between the width of ground improvements and factor of safety.
• For MSE walls, there is a linear relationship between the width of ground improvements and factor of safety up to the point at which \( B = D \), for \( D \leq 20 \text{ m} \).

• For engineered slopes, the ground improvements are most efficient when located between 25\%B inward and 50\%B outward, relative to the toe of slope.

• For MSE walls, the ground improvements are most efficient when located between 25\%B and 75\%B outward, relative to the toe of the wall.
2.6 References


3.0 Optimization of Sandwich Construction Using Re-claimed Dredging Waste as Backfill for Land Reclamation Applications

3.1 Introduction
Ground improvement methods such as deep soil mixing have been well established in Asia and Scandinavian Europe and are becoming increasingly common in North America as a means to remediate soft soils susceptible to excessive deformation, liquefaction, or having inadequate bearing capacity for the intended use (O’Rourke and McGinn 2006, Wooten et al. 2003, and Lambrechts et al. 2003).

Marine shipping facilities are a type of infrastructure which, based on their inherent locations, frequently need to be designed for undesirable soil conditions. Geomorphology of near shore deposits typically result in thick deposits of normally to slightly consolidated silts and clays which are susceptible to excessive settlements and slope instability. Increasingly, constraints due to limited available land and the fixed location of the proposed users of infrastructure mean that relocation to avoid difficult geotechnical conditions is not an acceptable alternative.

The use of deep soil mixing to provide adequate bearing capacity and global stability for large land reclamation projects and marine shipping facilities may be an acceptable method of addressing most of the geotechnical problems however; there remains the secondary problem of dealing with soft dredged materials. Dredged materials include material that is excavated at/or below the water level of channels, water basins or seas. Dredged materials are typically fine grained in nature and often are potentially
contaminated. Disposal of waste dredged materials may become cost prohibitive once the environmental mitigations and material costs are considered.

The purpose of this work is to evaluate the feasibility of re-using dredged or excavated soils as backfill in sandwich construction to reclaim land for future use. The goal of this study is to develop some general rules that may be used in developing a conceptual design for similar applications.

3.2 Related Research
The Japanese geotechnical community has embraced the use of mass soil stabilization as a means of reclaiming or constructing usable commercial property on their shorelines for several decades (CDIT 2002), and the practice has been adopted in other Southeast Asian countries as well as in Australia. A 580 ha man-made island was constructed for the Central Japan International Airport, in Nagoya, Japan using $70 \times 10^6$ m$^3$ of material. Of that, $8.2 \times 10^6$ m$^3$, or 12%, of the total material used was stabilized dredged soil; the remaining material was obtained from traditional terrestrial aggregate sources (Kitazume and Satoh 2003). Figure 3-1 below illustrates the different components required to complete pneumatic flow mixing used for the large scale reclamation.
The Changi East Reclamation Project at the Changi Airport in Singapore involved the reclamation of 2000 ha of land using $200 \times 10^6$ m$^3$ of sand. A cantilever wall, founded on soils improved by deep soil mixing was constructed and backfilled, as illustrated in Figure 3-2. The ground improvements varied between blocks and panels with replacement ratios of 50%, where the replacement ratio is the proportion of treated to untreated soil within a given volume.
A trial to explore the feasibility of using dredged soft clays for reclamation was also included as part of the work described above. The trial was completed using hydraulically placed materials and discovered constructability problems related to depositing sand layers on top of dredged clay slurry, and the time requirements between placing sand and clay layers (Choa 1994, and Choa et al. 2001).

A series of over two hundred counterfort and gravity walls, 20 m high, were used to complete a 63 ha reclamation for the Port Botany Expansion Project in Sydney, Australia. The reclamation was constructed using sand dredged from Botany Bay. As shown in Figure 3-3, a zone of clay was removed and replaced with compacted backfill within the footprint of the counterfort wall to meet stability and serviceability requirements. The project involved developing geotechnical compliance testing criteria to provide quality assurance of the reclamation fill materials so that the strength and stiffness of the in-place materials met design requirements (Davies and McIlquham 2011).
In England there has been consideration of reusing waste materials from maintenance dredging of navigation channels, ports, or marinas to restore coastlines as part of flood and coastal management work (Cooper 2004).

In addition to the marine applications noted above there are terrestrial applications to the re-use of soft excavated soils for large scale re-grading projects. Figure 3-4 shows an example where in order to maximize the usable footprint of a property material is excavated from the toe of the slope and placed at the top to construct a flat, usable area beyond the natural crest of the slope. An MSE wall, or alternative, would be required to satisfy global safety requirements; however, for the purpose of this research it was assumed to have an adequate factor of safety.
To date, there has been very little discussion or publication related to the optimization of the design of ground improvements within the footprint of a retaining structure. There has also been little discussion related to the use of dredged materials in sandwich construction for the purpose of reclamation, and if it is possible to optimize a design in terms of minimizing the waste of dredged materials and the consolidation settlements of the reclaimed land.

3.3 Problem Setup
A theoretical model was used to evaluate the sensitivity of the global factor of safety under a range of sandwich construction scenarios which included a retaining structure. The model was developed based on undesirable but common soil conditions; a thick layer
of soft, cohesive soil overlying bedrock. Analysis for this study was completed using a limit equilibrium software package (SlopeW), and Microsoft Excel.

Figure 3-5 illustrates the problem geometry used to evaluate a representative engineered slope. The engineered slope used for this analysis was selected based on results of analysis presented in Chapter 2. An average thickness of 20 m was selected for the soft, cohesive soil; the slope angle was observed to have negligible impact, therefore an average angle of 22° was used. It was determined that the strength of the mass stabilized ground improvements had a limited benefit after the strength exceeded 200 kPa; therefore the strength of ground improvements was constant at 200 kPa for this analysis. The ground improvements were located at a neutral position, where the outward edge of the ground improvements corresponded to the toe of the engineered slope.

![Diagram of representative cross section of problem setup](image)

Figure 3-5. Representative cross section of problem setup

The representative model was modified, as shown in Figure 3-6, to represent sandwich construction scenarios using alternating layers of reclaimed clay and granular backfill of variable proportions. A total of eight scenarios were evaluated as summarized in Table 3-1.
Figure 3-6. Typical cross section illustrating sandwich construction

Table 3-1. Summary of cases of sandwich construction used for analysis

<table>
<thead>
<tr>
<th>Case</th>
<th>Granular : Clay</th>
<th>Percentage of clay used as backfill for reclamation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1:0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>4:1</td>
<td>20</td>
</tr>
<tr>
<td>3</td>
<td>3:1</td>
<td>25</td>
</tr>
<tr>
<td>4</td>
<td>2:1</td>
<td>33</td>
</tr>
<tr>
<td>5</td>
<td>1:1</td>
<td>50</td>
</tr>
<tr>
<td>6</td>
<td>1:2</td>
<td>67</td>
</tr>
<tr>
<td>7</td>
<td>1:3</td>
<td>75</td>
</tr>
<tr>
<td>8</td>
<td>1:4</td>
<td>80</td>
</tr>
</tbody>
</table>
Case 1 is considered an idealized case where the total settlements of the reclaimed land would be minimized, however, would likely be economically prohibitive due to the cost of granular aggregate. The soil parameters used in the slope stability analyses are summarized in Table 3-2.

Table 3-2. Soil parameters used for slope stability analysis

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit Weight (kN/m$^3$)</th>
<th>$\Phi'$ (°)</th>
<th>$C_u$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>base sand</td>
<td>22.0</td>
<td>40</td>
<td>-</td>
</tr>
<tr>
<td>in-situ clay</td>
<td>15.0</td>
<td>-</td>
<td>25</td>
</tr>
<tr>
<td>embankment</td>
<td>19.6</td>
<td>35</td>
<td>-</td>
</tr>
<tr>
<td>reclaimed clay</td>
<td>14.5</td>
<td>-</td>
<td>10</td>
</tr>
<tr>
<td>ground improvements</td>
<td>15.0</td>
<td>-</td>
<td>200</td>
</tr>
</tbody>
</table>

The second phase of this study was to compare the anticipated total settlements associated with each of the cases described in Table 3-1. The example problem was assumed to be a two dimensional plane strain condition. It was assumed that the maximum settlements will occur behind the engineered slope and that settlements within the ground improvements would be negligible compared to adjacent reclaimed land located on un-remediated soils. The settlements were calculated adopting the following material properties presented in Table 3-3:
Table 3-3. Soil parameters used for settlement analysis

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit Weight (kN/m³)</th>
<th>E (MPa)</th>
<th>w (%)</th>
<th>e₀</th>
<th>cₑ</th>
<th>cᵥ   (cm²/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>in-situ clay</td>
<td>15.0</td>
<td>12.5</td>
<td>83</td>
<td>2.26</td>
<td>0.72</td>
<td>3.00E-03</td>
</tr>
<tr>
<td>embankment</td>
<td>19.6</td>
<td>85.0</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>reclaimed clay</td>
<td>14.5</td>
<td>5.0</td>
<td>95</td>
<td>2.54</td>
<td>0.82</td>
<td>1.00E-04</td>
</tr>
</tbody>
</table>

3.4 Results and Discussion

The following sections present results of the parametric study that was conducted to investigate the effect of width of ground improvements, percentage of clay used as backfill for reclamation, on the global factor of safety against slope failure. In addition, the impact of the use of reclaimed clay on the total settlement was investigated.

3.4.1 Optimizing the Width of Ground Improvements

In each of the eight cases, the width of the ground improvements were varied until the global factor of safety against circular shear failure was 1.50. It was observed that the cohesion in the reclaimed clay improved the overall global factor of safety and allowed for the width of the ground improvements at the toe of the slope be reduced. Figure 3-7 and Figure 3-8 shows Cases 1 and 8, respectively, where the width of ground improvements was reduced from 13.5 m to 11.2 m.
Figure 3-7. Optimization of ground improvements for Case 1, 100% granular backfill

Figure 3-8. Optimization of ground improvements for Case 8, 80% reclaimed clay and 20% granular backfill
Figure 3-9 plots the optimum width of ground improvements against the percentage of reclaimed clay used as backfill, Cases 1 through 8, respectively. As shown in the figure below, the relationship between the width of ground improvements is approximately linear, and not very sensitive, to the percentage of clay used as backfill. The figure shows a decrease in the minimum required width of ground improvements with the increase of the percentage of used dredged clay in the backfilling of the reclamation. This is due to the fact that the unit weight of the dredged clay is only 15 kN/m$^3$, which is almost 25% lighter than the unit weight of regular embankment fill (19.6 kN/m$^3$).

![Figure 3-9. Optimized widths of ground improvements](image)

3.4.2 Impact of Reclaimed Clay on Total Settlement

The total settlement is the sum of the elastic and consolidation (primary and secondary) settlements, as per equations 1 through 3.
Equation 1. Elastic deformation

\[ \delta_{\text{elastic}} = \frac{(\Delta \sigma)}{E} (H) \]

Where:
- \( \delta_{\text{elastic}} \) = elastic deformation
- \( \Delta \sigma \) = change in total stress
- \( E \) = Young’s Modulus
- \( H \) = thickness of soil unit

Equation 2. Primary consolidation settlement

\[ S_p = \frac{Cc}{1+e_o} (H_c) \log \left( \frac{\sigma_o' + \Delta \sigma}{\sigma_o'} \right) \]

Where:
- \( S_{\text{primary}} \) = Primary consolidation settlement
- \( c_c \) = coefficient of consolidation
- \( e_o \) = void ratio
- \( H_c \) = thickness of clay
- \( \Delta \sigma \) = change in total stress
- \( \sigma_o' \) = initial effective stress

Equation 3. Secondary consolidation settlement

\[ S_{\text{sec}} = \frac{C\alpha}{1+e_o} (H_c - S_p) \log \left( \frac{t_{\text{sec}}}{t_p} \right) \]

Where:
- \( S_{\text{sec}} \) = Secondary consolidation settlement;
- \( c_\alpha \) = coefficient of secondary consolidation
- \( e_o \) = void ratio
- \( H_c \) = thickness of clay
- \( S_{\text{primary}} \) = Primary consolidation settlement
- \( t_p \) = time to complete primary consolidation
- \( t_{\text{sec}} \) = time to complete secondary consolidation

The analysis was completed assuming a “gravity switch on” approach, or that the entire embankment was constructed instantaneously. This is not realistic; however, this approach provides upper bound estimates of settlement because it does not account for changes in material properties due to consolidation that takes place during the
construction. It was also assumed that the settlement within the footprint of the ground improvements would be negligible compared to those in untreated soils.

It was inherently anticipated that as the percentage of clay used as backfill increases that the total settlements will also increase. It was also anticipated that the minimum total settlements will occur with Case 1 (100% granular backfill). The total calculated settlements for Cases 1 through 8 are presented in Table 3-4.
Table 3-4. Results of settlement analysis

<table>
<thead>
<tr>
<th>case</th>
<th>% clay</th>
<th>Δσ (kPa)</th>
<th>δ&lt;sub&gt;emb&lt;/sub&gt; (m)</th>
<th>δ&lt;sub&gt;clay&lt;/sub&gt; (m)</th>
<th>S&lt;sub&gt;p&lt;/sub&gt; (m)</th>
<th>S&lt;sub&gt;sec&lt;/sub&gt; (m)</th>
<th>S&lt;sub&gt;reclaimed clay&lt;/sub&gt; (m)</th>
<th>S&lt;sub&gt;total&lt;/sub&gt; (m)</th>
<th>%Δ S&lt;sub&gt;total&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>196</td>
<td>0.012</td>
<td>0.314</td>
<td>3.00</td>
<td>0.105</td>
<td>0.000</td>
<td>3.43</td>
<td>0%</td>
</tr>
<tr>
<td>2</td>
<td>20</td>
<td>187</td>
<td>0.013</td>
<td>0.299</td>
<td>2.92</td>
<td>0.106</td>
<td>0.594</td>
<td>3.94</td>
<td>15%</td>
</tr>
<tr>
<td>3</td>
<td>25</td>
<td>185</td>
<td>0.014</td>
<td>0.295</td>
<td>2.91</td>
<td>0.106</td>
<td>0.793</td>
<td>4.11</td>
<td>20%</td>
</tr>
<tr>
<td>4</td>
<td>33</td>
<td>181</td>
<td>0.015</td>
<td>0.289</td>
<td>2.87</td>
<td>0.106</td>
<td>1.046</td>
<td>4.33</td>
<td>26%</td>
</tr>
<tr>
<td>5</td>
<td>50</td>
<td>173</td>
<td>0.018</td>
<td>0.277</td>
<td>2.81</td>
<td>0.107</td>
<td>1.266</td>
<td>4.48</td>
<td>31%</td>
</tr>
<tr>
<td>6</td>
<td>67</td>
<td>165</td>
<td>0.023</td>
<td>0.265</td>
<td>2.74</td>
<td>0.107</td>
<td>1.760</td>
<td>4.90</td>
<td>43%</td>
</tr>
<tr>
<td>7</td>
<td>75</td>
<td>162</td>
<td>0.026</td>
<td>0.258</td>
<td>2.71</td>
<td>0.107</td>
<td>1.637</td>
<td>4.74</td>
<td>38%</td>
</tr>
<tr>
<td>8</td>
<td>80</td>
<td>159</td>
<td>0.029</td>
<td>0.255</td>
<td>2.69</td>
<td>0.107</td>
<td>1.534</td>
<td>4.61</td>
<td>35%</td>
</tr>
</tbody>
</table>

It is noted that the maximum total settlement corresponds to Case 6 (67% clay) rather than Case 8 (80% clay). This observation is attributed to the differences in unit weight between the granular aggregate and clay backfill, 19.6 kN/m³ and 14.5 kN/m³, respectively. As the %clay increases the total stress applied due to the construction of the embankment decreases, which results in a reduction of primary settlement. The increased volume of clay within the embankment results in increased consolidation settlement within the embankment itself, however, due to the difference in unit weights of the materials the same observation applies with respect to the applied stress where the maximum S<sub>reclaimed clay</sub> corresponds to 67% clay.
Figure 3-10 provides a direct comparison between the percentage reduction in the width of ground improvements and the corresponding percentage increase in settlements associated with the sandwich construction scenarios. As described above there is a distinct optimization in terms of the settlement where 67% clay (or a 1:2 granular to clay ratio) resulted in the maximum settlement. It was also observed that the impact on settlement was far greater than the impact on width of ground improvements between similar sandwich construction scenarios.

![Figure 3-10. Tradeoff comparison between settlement and widths of ground improvements for sandwich construction](image)

3.5 Conclusions
Based on the analysis completed, the following conclusions have been drawn regarding the re-use of dredged/excavated clay materials as backfill in sandwich construction for land reclamation applications.

- The cohesion provided by the dredged clay materials provides opportunity for further optimization of the width of ground improvements.
Total settlement increases as the proportion of dredged clay materials increases, to a maximum which occurs at approximately a granular to clay layer ratio of 1:2.

The greater the difference between the unit weights of the dredged clay and the granular aggregate is, the more pronounced the variance of total settlements will be.

For the purpose of optimization of design by re-using dredged clay, the benefit of reducing the width of ground improvements is limited compared to the corresponding increase in anticipated total settlements.
3.6 References


4.0 Summary and Conclusions

Deep soil mixing technology is a tool that may potentially make some land reclamation projects technically, and financially, feasible. Addressing global stability problems by mixing in situ soils with binders such as cement or lime can allow for previously unsuitable sites to become commercially useful. There may be financial and environmental benefits to reusing dredged soils in backfilling, however, there are tradeoffs related to serviceability of the site to be considered during such a design.

The research work for this thesis was completed using a combination of slope stability software and hand calculations. The problem was based on theoretical, but realistic, site conditions. Literature review was completed to determine relevant material properties to be used in the analysis, as well as review of recent applications of deep soil mixing technology.

This project had two primary components; (i) optimization of the design of ground improvements for both engineered slopes and MSE walls; (ii) quantifying the potential benefits of reusing dredged or excavated soft soils to reclaim land or site re-grading. The strength, width, and locations of ground improvements were considered for variable thicknesses of soft, cohesive soils to determine the global factor of safety. These parameters were also analyzed under variable engineered slope angles. The second component of this project was completed using a representative cross section based on
the findings of the first part of the study. The problem may be represented using two-dimensional plane strain analysis; therefore hand calculations were used.

The objective of this work was to determine which parameters may be used to optimize the design of ground improvements as well as to determine if there was potential for optimization of a case where dredged or excavated soft soils can be used to reclaim/regrade land behind retaining structures located on improved ground.

Based on the analysis completed the following conclusions have been drawn regarding the design of ground improvements for global stability of engineered slopes and MSE walls:

- The minimum strength of improved ground needs to be 100 kPa.
- The benefit of the strength of improved ground becomes negligible after strength exceeds 200 kPa.
- For engineered slopes, the slope angle has negligible impact to the overall factor of safety.
- For engineered slopes, there is a linear relationship between the width of ground improvements and the factor of safety.
- For engineered slopes, the ground improvements are most efficient when located between 25%B inward and 50%B outward, relative to the toe of slope.
- For MSE walls, there is a linear relationship between the width of ground improvements and factor of safety up to the point at which $B = D$, for $D \leq 20$ m.
• For MSE walls, the ground improvements are most efficient when located between 25%B₁ and 75%B outward, relative to the toe of the wall.

Based on the analysis completed the following conclusions have been drawn regarding the re-use of dredged/excavated soft clay materials as backfill in sandwich construction for land reclamation/site re-grading applications:

• The cohesion provided by the dredged clay materials provides opportunity for further optimization of the width of ground improvements.

• Total settlement increase as the proportion of dredged clay materials increases, to a maximum which occurs at approximately a granular to clay layer ratio of 1:2.

• The greater the difference between the unit weights of the dredged clay and the granular aggregate the more pronounced the differences in total settlements will be.

• For the purpose of optimization of design by re-using dredged/excavated clay; the benefit of reducing the width of ground improvements is limited compared to the corresponding increase in anticipated total settlements.
5.0 Recommendations for Future Research

A number of questions which require additional attention have evolved during completion of this work. The problem was based on theoretical soil parameters which were kept constant throughout the parametric study. It is unknown what impact the in-situ strength of the cohesive soil has on the design of ground improvements. It is anticipated that as the in-situ strength increased or decreased that the minimum width of ground improvements would decrease and increase, respectively, however, it is unknown if the magnitude of the changes in the minimum widths would be proportional to the change in strength. A second parameter which was kept constant throughout the work was the height of the embankment/reclaimed land. It is unknown if the parameters considered in this work would have the same limitations or sensitivity to changes in either the soil strengths or heights of embankments.

The analyses completed for this research also assumed that the ground improvements consisted of homogenous, mass stabilization. In much of the literature there is discussion of ground improvements in the forms of columns arranged in a variety of layouts ranging between individual columns, or panels constructed of overlapping columns, and grids formed by intersecting panels. It would be useful to know if the findings from this research are consistent for ground improvements regardless of their installation layout. Due to the three dimensional nature of the columns, panels, or grids a more sophisticated numerical modeling software would have to be used to accurately capture the global behavior. A finite difference numerical modeling software would be recommended,
particularly due to findings published by Navin (2005) where it has been observed that more traditional two dimensional finite element software is non-conservative because shear failure is not the governing mode of failure for columns, or panels of ground improvements.

Further, for the research related to the use of dredged/excavated soil for land reclamation or site re-grading, it was conservatively assumed that the material properties of the reclaimed soil did not change during consolidation which would occur during construction and loading of the soils. Consideration of changes in material properties in a detailed analysis may result in different ranges of responses or sensitivities.
6.0 References


Curriculum Vitae

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