FINITE ELEMENT STUDY ON
STATIC PILE LOAD TESTING

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(B.Eng)

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Dedicated to my family and friends
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# TABLE OF CONTENTS

ACKNOWLEDGEMENTS............................................................................................. i
TABLE OF CONTENTS................................................................................................ ii
SUMMARY ................................................................................................................... iv
LIST OF TABLES ......................................................................................................... vi
LIST OF FIGURES ...................................................................................................... vii
LIST OF SYMBOLS ..................................................................................................... xi

CHAPTER 1 INTRODUCTION .................................................................................... 1  
1.1 Objectives ....................................................................................................... 1  
1.2 Scope of Study ................................................................................................ 3  

CHAPTER 2 LITERATURE REVIEW ......................................................................... 5  
2.1 Review of Pile Load Test................................................................................ 5  
2.2 Reaction System and Static Load Test............................................................ 6  
  2.2.1 Recommended Distance of Reaction System for Static Load Test ... 6  
  2.2.2 Interaction Effect of Reaction System on the Results of Static load Test.................................................................................. 9  
2.3 Comparison of O-Cell Test with Static Load Test........................................ 14  
2.4 Finite Element Analysis................................................................................ 17  
  2.4.1 Review of Theoretical Method ......................................................... 17  
  2.4.2 Introduction to PLAXIS and PLAXIS 3D Foundation..................... 19  

CHAPTER 3 FEM STUDY ON EFFECT OF REACTION SYSTEM ....................... 46  
3.1 Introduction................................................................................................... 46  
3.2 Pile Load Test with Kentledge...................................................................... 50  
  3.2.1 General.............................................................................................. 50  
  3.2.2 Influence of L/D................................................................................ 51  
  3.2.3 Influence of B ................................................................................... 51  
  3.2.4 Influence of Area of Cribbage .......................................................... 52  
  3.2.5 Influence of K ................................................................................... 52  
3.3 Pile Load Test with Tension Piles ................................................................ 53  
  3.3.1 General.............................................................................................. 53  
  3.3.2 Influence of L/D................................................................................ 54  
  3.3.3 Influence of D ................................................................................... 55  
  3.3.4 Influence of Load Level.................................................................... 55  
  3.3.5 Influence of K ................................................................................... 56  
3.4 Conclusions................................................................................................... 57

CHAPTER 4 ................................................................................................................. 66  
FEM STUDY ON O-CELL TEST ............................................................................... 66  
4.1 Methodology.................................................................................................. 66  
  4.1.1 Introduction........................................................................................ 66  
  4.1.2 Construction of the Equivalent Head-down Load-Settlement Curve68  
  4.1.3 Elastic Compression......................................................................... 69  
4.2 Shaft Resistance Comparison....................................................................... 70  
  4.2.1 Load Transfer Curve......................................................................... 71  
  4.2.2 Unit Shaft Resistance........................................................................ 72  
  4.2.3 t-z Curve............................................................................................ 73  
4.3 End Bearing Comparison.............................................................................. 75  
4.4 Equivalent Head-down Load-Movement Curve........................................... 76  
4.5 Drained Analysis........................................................................................... 77
SUMMARY

Pile load test is a fundamental part of pile foundation design. Although many pile tests have been constructed in all kinds of engineering projects, it is unclear what difference arises from newer test methods such as the O-cell test. An accurate interpretation of the pile test would be difficult unless some aspects such as whether the different types of load test or test set-up may have any side-effects on the test results is clearly understood.

In this thesis, the finite element method (FEM) was used to carry out the research. The commercial finite element code PLAXIS and PLAXIS 3D Foundation were used for the numerical simulation of pile load test in the following manner.

The thesis focuses on some particular interest which is associated with the conventional static load test and Osterberg-cell test. Different reaction systems for the static pile load test are analyzed to study the effect of reaction system on the test results. The numerical results indicate that the influence of the reaction system on the settlement of the test pile is always under-estimated in practice. The commonly recommended minimum spacing of 3D–5D between test pile and reaction system may not be enough, as it tends to have greater influence on test pile results than desired. Other parameters that are involved such as L/D ratio, D, Diameter of reaction piles, B, the width of the cribbage, the area of the cribbage, E_{pile}/E_{soil}, load level etc. are studied and correction factor F_c vs. S/D ratio relation are illustrated.

Furthermore, O-cell test is compared with static pile load test and equivalency and
discrepancy of the test results between the two types of pile load test are demonstrated and analyzed. It is concluded that O-cell test result can provide not only the same soil-pile interaction information as conventional head-down static loading test, but also allow for separate determination of the shaft resistance and end bearing components. However, the equivalent head down load-movement curve of the O-cell test simulated by PLAXIS 8 gives a slightly stiffer load-movement response and slightly higher ultimate capacity than those of conventional test. The differences of effective stresses around the pile due to the different excess pore pressures generated from the different load-transfer mechanism of these two kinds of pile load tests contributed to the discrepancy of unit shaft resistance of these test piles under the same pile movement. When drained analyses were made and long-term soil-pile interaction was considered, both the O-cell test and conventional test gave nearly identical results.

Keywords: Pile load test, FEM, PLAXIS, Conventional static load test, Reaction system, Osterberg load test.
## LIST OF TABLES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table 2.1</td>
<td>Recommended Spacing between Test Pile and Reaction System</td>
<td>8</td>
</tr>
<tr>
<td>Table 3.1</td>
<td>Basic Geometrical properties of 3D Models</td>
<td>47</td>
</tr>
<tr>
<td>Table 3.2a</td>
<td>Material properties used in the analyses</td>
<td>47</td>
</tr>
<tr>
<td>Table 3.2b</td>
<td>Material properties used in the analyses</td>
<td>48</td>
</tr>
<tr>
<td>Table 4.1</td>
<td>Geometrical properties of mesh and structure</td>
<td>67</td>
</tr>
<tr>
<td>Table 4.2</td>
<td>Material properties of the FEM model</td>
<td>67</td>
</tr>
<tr>
<td>Table 5.1</td>
<td>Average Net Unit Shaft Resistance for 1L-34</td>
<td>96</td>
</tr>
<tr>
<td>Table 5.2</td>
<td>Material Properties of PTP1 in PLAXIS 8</td>
<td>97</td>
</tr>
<tr>
<td>Table 6.1</td>
<td>Soil and concrete properties</td>
<td>119</td>
</tr>
<tr>
<td>Table 6.2</td>
<td>Material properties of NTUC</td>
<td>126</td>
</tr>
<tr>
<td>Table 6.3</td>
<td>Soil properties of NTUC</td>
<td>126</td>
</tr>
<tr>
<td>Figure</td>
<td>Title</td>
<td>Page</td>
</tr>
<tr>
<td>--------</td>
<td>------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>Fig. 2.1</td>
<td>Schematic Set-Up for Static Pile Loading Test Using Kentledge</td>
<td>30</td>
</tr>
<tr>
<td>Fig. 2.2</td>
<td>Schematic Set-Up for Static Pile Loading Test Using Anchored Reaction Piles</td>
<td>31</td>
</tr>
<tr>
<td>Fig. 2.3</td>
<td>Schematic Set-Up for Static Pile Loading Test Using Ground Anchor</td>
<td>32</td>
</tr>
<tr>
<td>Fig. 2.4</td>
<td>Schematic Set-Up for Osterberg-Cell Test</td>
<td>33</td>
</tr>
<tr>
<td>Fig. 2.5</td>
<td>Plan with Location of CPT and 6 Anchor-piles</td>
<td>34</td>
</tr>
<tr>
<td>Fig. 2.6</td>
<td>Result of 2 Load Tests on the Same Pile</td>
<td>34</td>
</tr>
<tr>
<td>Fig. 2.7</td>
<td>Comparison of Total Load, Skin Friction and Tip Resistance</td>
<td>35</td>
</tr>
<tr>
<td>Fig. 2.8</td>
<td>Comparison of Skin Friction with Settlement of the Test Piles</td>
<td>35</td>
</tr>
<tr>
<td>Fig. 2.9</td>
<td>Development of the Influence Factors with Settlements</td>
<td>36</td>
</tr>
<tr>
<td>Fig. 2.10</td>
<td>Example of Influence of Kentledge on Pile Test in Sand</td>
<td>36</td>
</tr>
<tr>
<td>Fig. 2.11</td>
<td>Correction Factor $F_c$ for Floating Pile in a Deep Layer Jacked against Two Reaction Piles</td>
<td>37</td>
</tr>
<tr>
<td>Fig. 2.12</td>
<td>Correction Factor $F_c$ for End-bearing Pile on Rigid Stratum Jacked against Two Reaction Piles</td>
<td>37</td>
</tr>
<tr>
<td>Fig. 2.13</td>
<td>Comparison of Circular Footing and Strip Footing, When B=1m, 2m and 2.5m</td>
<td>38</td>
</tr>
<tr>
<td>Fig. 2.14</td>
<td>Comparison of Circular Footing and Strip Footing with Different $C_u$ Values</td>
<td>38</td>
</tr>
<tr>
<td>Fig. 2.15</td>
<td>Interaction Factor Ratio $\beta$ for London Clay</td>
<td>39</td>
</tr>
<tr>
<td>Fig. 2.16</td>
<td>Interaction Factor Ratio $\beta$ for London Clay</td>
<td>40</td>
</tr>
<tr>
<td>Fig. 2.17</td>
<td>Comparison of the Deflection-end Bearing Curve of O-cell and Top Down Test</td>
<td>41</td>
</tr>
<tr>
<td>Fig. 2.18</td>
<td>Comparison of the Load-Movement Curve of Measured and Calculated</td>
<td>41</td>
</tr>
<tr>
<td>Fig. 2.19</td>
<td>Comparison of the Shaft Resistance Value</td>
<td>42</td>
</tr>
<tr>
<td>Fig. 2.20</td>
<td>Theoretical Comparison Between Ideal Tests and O-cell Test for Pile in Sand</td>
<td>43</td>
</tr>
<tr>
<td>Fig. 2.21</td>
<td>Vertical Load versus Depth for O-cell and Head test</td>
<td>44</td>
</tr>
<tr>
<td>Fig. 2.22</td>
<td>Unit Side Shear versus Depth for O-cell and Head Test</td>
<td>44</td>
</tr>
<tr>
<td>Figure</td>
<td>Title</td>
<td>Page</td>
</tr>
<tr>
<td>----------</td>
<td>----------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>Fig. 2.23</td>
<td>Load-Movement for Equivalent Head-Down Test</td>
<td>45</td>
</tr>
<tr>
<td>Fig. 2.24</td>
<td>Hyperbolic Stress-strain Relations in Primary Loading in Standard Drained Triaxial Test</td>
<td>45</td>
</tr>
<tr>
<td>Fig.3.1</td>
<td>Geometric Parameters of 3D Model</td>
<td>59</td>
</tr>
<tr>
<td>Fig.3.2a</td>
<td>3D Model of Kentledge System</td>
<td>60</td>
</tr>
<tr>
<td>Fig.3.2b</td>
<td>3D Model of Reaction Pile System</td>
<td>60</td>
</tr>
<tr>
<td>Fig.3.3</td>
<td>Influence of L/D – Kentledge System</td>
<td>61</td>
</tr>
<tr>
<td>Fig.3.4</td>
<td>Influence of B – Kentledge System</td>
<td>61</td>
</tr>
<tr>
<td>Fig.3.5</td>
<td>Influence of Area of Cribbage – Kentledge System</td>
<td>62</td>
</tr>
<tr>
<td>Fig.3.6</td>
<td>Influence of K – Kentledge System</td>
<td>62</td>
</tr>
<tr>
<td>Fig.3.7</td>
<td>Influence of L/D - Reaction Pile System</td>
<td>63</td>
</tr>
<tr>
<td>Fig.3.8</td>
<td>Influence of Diameter of Reaction Pile System</td>
<td>63</td>
</tr>
<tr>
<td>Fig.3.9</td>
<td>Influence of Load Level - Reaction Pile System</td>
<td>64</td>
</tr>
<tr>
<td>Fig.3.10</td>
<td>Influence of Load Level - Reaction Pile System</td>
<td>64</td>
</tr>
<tr>
<td>Fig.3.11</td>
<td>Influence of K - Reaction Pile System</td>
<td>65</td>
</tr>
<tr>
<td>Fig.4.1</td>
<td>FEM Model of Bottom O-cell Test</td>
<td>81</td>
</tr>
<tr>
<td>Fig.4.2</td>
<td>FEM Model of Middle O-cell Test</td>
<td>82</td>
</tr>
<tr>
<td>Fig.4.3</td>
<td>FEM Model of Conventional Static Pile load Test</td>
<td>83</td>
</tr>
<tr>
<td>Fig.4.4</td>
<td>Calculation of Elastic Compression using Triangular Side Shear Distribution</td>
<td>84</td>
</tr>
<tr>
<td>Fig.4.5</td>
<td>Comparison of Load-Transfer Curves</td>
<td>84</td>
</tr>
<tr>
<td>Fig.4.6</td>
<td>Comparison of Unit Shaft Resistance Curves</td>
<td>85</td>
</tr>
<tr>
<td>Fig.4.7</td>
<td>Comparison of t-z Curves at EL.10m</td>
<td>85</td>
</tr>
<tr>
<td>Fig.4.8</td>
<td>Comparison of t-z Curves at EL. 19m</td>
<td>86</td>
</tr>
<tr>
<td>Fig.4.9</td>
<td>Comparison of End-Bearing Curves</td>
<td>86</td>
</tr>
<tr>
<td>Fig.4.10</td>
<td>Comparison of Load-Movement Curves (Rigid Pile)</td>
<td>87</td>
</tr>
<tr>
<td>Fig.4.11</td>
<td>Comparison of Load-Movement Curves (Flexible Pile)</td>
<td>87</td>
</tr>
<tr>
<td>Fig.4.12</td>
<td>Comparison of Load-Transfer Curves (Drained)</td>
<td>88</td>
</tr>
<tr>
<td>Fig.4.13</td>
<td>Comparison of Unit Shaft Resistance Curves (Drained)</td>
<td>88</td>
</tr>
<tr>
<td>Fig.4.14</td>
<td>Comparison of Load-Transfer Curves (Drained)</td>
<td>89</td>
</tr>
<tr>
<td>Fig.5.1</td>
<td>Location of Case Study in Gopeng Street</td>
<td>105</td>
</tr>
<tr>
<td>Figure</td>
<td>Title</td>
<td>Page</td>
</tr>
<tr>
<td>---------</td>
<td>----------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>Fig.5.2</td>
<td>Instrumentation of PTP1</td>
<td>105</td>
</tr>
<tr>
<td>Fig. 5.3</td>
<td>FEM Model of PTP1</td>
<td>106</td>
</tr>
<tr>
<td>Fig.5.4</td>
<td>Adhesion Factors for Bored Pile (after Weltman and Healy )</td>
<td>107</td>
</tr>
<tr>
<td>Fig.5.5</td>
<td>Plate Loading Test by Duncan and Buchignani (1976)</td>
<td>107</td>
</tr>
<tr>
<td>Fig.5.6</td>
<td>Comparison of Load-Movement Curve</td>
<td>108</td>
</tr>
<tr>
<td>Fig.5.7</td>
<td>Comparison of Load-Transfer Curve at 1L-8</td>
<td>108</td>
</tr>
<tr>
<td>Fig.5.8</td>
<td>Comparison of Load-Transfer Curve at 1L-16</td>
<td>109</td>
</tr>
<tr>
<td>Fig.5.9</td>
<td>Comparison of Load-Transfer Curve at 1L-24</td>
<td>109</td>
</tr>
<tr>
<td>Fig.5.10</td>
<td>Comparison of Load-Transfer Curve at 1L-34</td>
<td>110</td>
</tr>
<tr>
<td>Fig.5.11</td>
<td>Comparison of Unit Shaft Resistance of Curve at 1L-8</td>
<td>110</td>
</tr>
<tr>
<td>Fig.5.12</td>
<td>Comparison of Unit Shaft Resistance of Curve at 1L-16</td>
<td>111</td>
</tr>
<tr>
<td>Fig.5.13</td>
<td>Comparison of Unit Shaft Resistance of Curve at 1L-24</td>
<td>111</td>
</tr>
<tr>
<td>Fig.5.14</td>
<td>Comparison of Unit Shaft Resistance of Curve at 1L-34</td>
<td>112</td>
</tr>
<tr>
<td>Fig.5.15</td>
<td>Extrapolation of Load-Movement Curve by FEM</td>
<td>112</td>
</tr>
<tr>
<td>Fig.5.16</td>
<td>Comparison of Load-Transfer Curve of O-cell at 1L-34 with</td>
<td>113</td>
</tr>
<tr>
<td></td>
<td>That of Equivalent Conventional Test</td>
<td></td>
</tr>
<tr>
<td>Fig.5.17</td>
<td>Comparison of Unit Shaft Resistance Curve of O-cell at 1L-34 with</td>
<td>113</td>
</tr>
<tr>
<td></td>
<td>That of Equivalent Conventional Test</td>
<td></td>
</tr>
<tr>
<td>Fig.5.18</td>
<td>Equivalent Top Load-Movement Curves</td>
<td>114</td>
</tr>
<tr>
<td>Fig.5.19</td>
<td>Comparison of Distribution of Excess Pore Pressure</td>
<td>115</td>
</tr>
<tr>
<td>Fig.5.20</td>
<td>Comparison of Distribution of Effective Normal Stress</td>
<td>115</td>
</tr>
<tr>
<td>Fig.6.1</td>
<td>Pile Load Arrangement and Design Soil Profile</td>
<td>133</td>
</tr>
<tr>
<td>Fig.6.2</td>
<td>3D FEM Model with Four Reaction Piles</td>
<td>134</td>
</tr>
<tr>
<td>Fig.6.3</td>
<td>Load-Settlement Curve of 4 Reaction Piles System</td>
<td>135</td>
</tr>
<tr>
<td>Fig.6.4</td>
<td>Comparison of Load-Settlement Curve of 4 Reaction Piles System with</td>
<td>135</td>
</tr>
<tr>
<td></td>
<td>Single Pile</td>
<td></td>
</tr>
<tr>
<td>Fig.6.5</td>
<td>3D FEM Model with Two Reaction Piles</td>
<td>136</td>
</tr>
<tr>
<td>Fig.6.6</td>
<td>Comparison of Load-Settlement Curve of 4 Reaction Piles System with</td>
<td>137</td>
</tr>
<tr>
<td></td>
<td>2 Reaction Piles</td>
<td></td>
</tr>
<tr>
<td>Fig.6.7</td>
<td>Influence of Different Numbers of Reaction Piles</td>
<td>137</td>
</tr>
<tr>
<td>Fig.6.8</td>
<td>Location of Instruments in Test Pile of NTUC</td>
<td>138</td>
</tr>
<tr>
<td>Fig.6.9</td>
<td>FEM Model of NTUC</td>
<td>139</td>
</tr>
<tr>
<td>Figure</td>
<td>Title</td>
<td>Page</td>
</tr>
<tr>
<td>---------</td>
<td>-------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>Fig.6.10</td>
<td>Load-Movement Curve</td>
<td>140</td>
</tr>
<tr>
<td>Fig.6.11</td>
<td>Load-Transfer Curve at 1×W.L.</td>
<td>140</td>
</tr>
<tr>
<td>Fig.6.12</td>
<td>Load-Transfer Curve at 2×W.L.</td>
<td>141</td>
</tr>
<tr>
<td>Fig.6.13</td>
<td>Load-Transfer Curve at 3×W.L.</td>
<td>141</td>
</tr>
<tr>
<td>Fig.6.14</td>
<td>Unit Shaft Resistance Curve at 1×W.L.</td>
<td>142</td>
</tr>
<tr>
<td>Fig.6.15</td>
<td>Unit Shaft Resistance Curve at 2×W.L.</td>
<td>142</td>
</tr>
<tr>
<td>Fig.6.16</td>
<td>Unit Shaft Resistance Curve at 3×W.L.</td>
<td>143</td>
</tr>
<tr>
<td>Fig.6.17</td>
<td>Comparison of Load-Movement Curve</td>
<td>143</td>
</tr>
<tr>
<td>Fig.6.18</td>
<td>Comparison of Load-Transfer Curve at 1×W.L.</td>
<td>144</td>
</tr>
<tr>
<td>Fig.6.19</td>
<td>Comparison of Load-Transfer Curve at 2×W.L.</td>
<td>144</td>
</tr>
<tr>
<td>Fig.6.20</td>
<td>Comparison of Load-Transfer Curve at 3×W.L.</td>
<td>145</td>
</tr>
<tr>
<td>Fig.6.21</td>
<td>Comparison of Unit Shaft Resistance Curve at 1×W.L.</td>
<td>145</td>
</tr>
<tr>
<td>Fig.6.22</td>
<td>Comparison of Unit Shaft Resistance Curve at 2×W.L.</td>
<td>146</td>
</tr>
<tr>
<td>Fig.6.23</td>
<td>Comparison of Unit Shaft Resistance Curve at 3×W.L.</td>
<td>146</td>
</tr>
<tr>
<td>Fig.6.24</td>
<td>Comparison of Shaft and End Bearing Resistance vs. Movement curve</td>
<td>147</td>
</tr>
<tr>
<td>Symbol</td>
<td>Units</td>
<td>Meaning</td>
</tr>
<tr>
<td>--------</td>
<td>-------</td>
<td>---------</td>
</tr>
<tr>
<td>B</td>
<td>m</td>
<td>Width of cribbage</td>
</tr>
<tr>
<td>CPT</td>
<td></td>
<td>Cone penetration test</td>
</tr>
<tr>
<td>c</td>
<td>kN/m²</td>
<td>Cohesion</td>
</tr>
<tr>
<td>c&lt;sub&gt;actual&lt;/sub&gt;</td>
<td>kN/m²</td>
<td>Actual cohesion</td>
</tr>
<tr>
<td>c&lt;sub&gt;i&lt;/sub&gt;</td>
<td>kN/m²</td>
<td>Cohesion of interface element</td>
</tr>
<tr>
<td>c&lt;sub&gt;increment&lt;/sub&gt;</td>
<td>kN/m²</td>
<td>The increase of cohesion per unit depth</td>
</tr>
<tr>
<td>c&lt;sub&gt;soil&lt;/sub&gt;</td>
<td>kN/m²</td>
<td>Cohesion of soil</td>
</tr>
<tr>
<td>c&lt;sub&gt;u&lt;/sub&gt;</td>
<td>kN/m²</td>
<td>Undrained shear strength</td>
</tr>
<tr>
<td>d</td>
<td>m</td>
<td>Diameter of pile or thickness of cribbage</td>
</tr>
<tr>
<td>D</td>
<td>m</td>
<td>Diameter of pile</td>
</tr>
<tr>
<td>E</td>
<td>MN/m²</td>
<td>Young’s modulus</td>
</tr>
<tr>
<td>E&lt;sub&gt;50&lt;/sub&gt;</td>
<td>MN/m²</td>
<td>Confining stress-dependent stiffness modulus for primary loading</td>
</tr>
<tr>
<td>E&lt;sub&gt;50&lt;/sub&gt;&lt;sup&gt;ref&lt;/sup&gt;</td>
<td>MN/m²</td>
<td>Reference stiff modulus corresponding to the reference confining pressure</td>
</tr>
<tr>
<td>EA</td>
<td>kN/m</td>
<td>Elastic axial stiffness</td>
</tr>
<tr>
<td>EI</td>
<td>kN.m²/m</td>
<td>Bending stiffness</td>
</tr>
<tr>
<td>E&lt;sub&gt;actual&lt;/sub&gt;</td>
<td>MN/m²</td>
<td>Actual Young’s modulus</td>
</tr>
<tr>
<td>E&lt;sub&gt;i&lt;/sub&gt;</td>
<td>MN/m²</td>
<td>Young’s modulus of interface element</td>
</tr>
<tr>
<td>E&lt;sub&gt;increment&lt;/sub&gt;</td>
<td>MN/m²</td>
<td>The increase of the Young’s modulus per unit of depth</td>
</tr>
<tr>
<td>E&lt;sub&gt;ref&lt;/sub&gt;</td>
<td>MN/m²</td>
<td>Reference Young’s modulus</td>
</tr>
<tr>
<td>E&lt;sub&gt;s&lt;/sub&gt;/E&lt;sub&gt;soil&lt;/sub&gt;</td>
<td>MN/m²</td>
<td>Young’s modulus of soil</td>
</tr>
<tr>
<td>E&lt;sub&gt;p&lt;/sub&gt;</td>
<td>MN/m²</td>
<td>Young’s modulus of pile</td>
</tr>
<tr>
<td>E&lt;sub&gt;oed&lt;/sub&gt;</td>
<td>MN/m²</td>
<td>Constrained or oedometric soil modulus</td>
</tr>
<tr>
<td>E&lt;sub&gt;oed&lt;/sub&gt;&lt;sup&gt;ref&lt;/sup&gt;</td>
<td>MN/m²</td>
<td>Tangent stiffness for primary oedometer loading</td>
</tr>
<tr>
<td>E&lt;sub&gt;ur&lt;/sub&gt;&lt;sup&gt;ref&lt;/sup&gt;</td>
<td>MN/m²</td>
<td>Reference Young’s modulus for unloading/reloading</td>
</tr>
<tr>
<td>F&lt;sub&gt;c&lt;/sub&gt;</td>
<td>MN/m²</td>
<td>Correction factors of pile settlement</td>
</tr>
<tr>
<td>FEM</td>
<td></td>
<td>Finite element method</td>
</tr>
<tr>
<td>G</td>
<td>MN/m²</td>
<td>Shear modulus</td>
</tr>
<tr>
<td>Symbol</td>
<td>Units</td>
<td>Meaning</td>
</tr>
<tr>
<td>--------</td>
<td>-------</td>
<td>---------</td>
</tr>
<tr>
<td>H</td>
<td>m</td>
<td>Height of soil profile</td>
</tr>
<tr>
<td>K</td>
<td></td>
<td>Pile stiffness factor</td>
</tr>
<tr>
<td>K’</td>
<td>MN/m$^2$</td>
<td>Effective bulk modulus</td>
</tr>
<tr>
<td>$K_w$</td>
<td>MN/m$^2$</td>
<td>Bulk modulus of water</td>
</tr>
<tr>
<td>$K_o$</td>
<td></td>
<td>Coefficient of lateral stress in in-situ condition</td>
</tr>
<tr>
<td>$K_o^{NC}$</td>
<td></td>
<td>Coefficient of lateral stress in normal consolidation</td>
</tr>
<tr>
<td>L</td>
<td>m</td>
<td>Length of Pile</td>
</tr>
<tr>
<td>$l_e$</td>
<td>m</td>
<td>Average element size</td>
</tr>
<tr>
<td>m</td>
<td></td>
<td>Power in stress-dependent stiffness relation</td>
</tr>
<tr>
<td>n</td>
<td></td>
<td>Porosity</td>
</tr>
<tr>
<td>OCR</td>
<td></td>
<td>Over consolidation ratio</td>
</tr>
<tr>
<td>$p_{ref}$</td>
<td>kN/m$^2$</td>
<td>Reference confining pressure</td>
</tr>
<tr>
<td>Q</td>
<td>kN</td>
<td>Total load</td>
</tr>
<tr>
<td>$Q_s$</td>
<td>kN</td>
<td>Shaft resistance</td>
</tr>
<tr>
<td>$Q_t$</td>
<td>kN</td>
<td>Tip resistance or end bearing</td>
</tr>
<tr>
<td>$q_a$</td>
<td>kN/m$^2$</td>
<td>Asymptotic value of the shear strength</td>
</tr>
<tr>
<td>$q_c$</td>
<td>kN/m$^2$</td>
<td>Average cone resistance</td>
</tr>
<tr>
<td>$q_f$</td>
<td>kN/m$^2$</td>
<td>Ultimate deviatoric stress</td>
</tr>
<tr>
<td>$q_s$</td>
<td>kN/m$^2$</td>
<td>Ultimate shaft resistance</td>
</tr>
<tr>
<td>$R_f$</td>
<td></td>
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<td>$R_{inter}$</td>
<td></td>
<td>Interface strength reduction factor</td>
</tr>
<tr>
<td>r</td>
<td>m</td>
<td>Distance from the center of footing</td>
</tr>
<tr>
<td>S</td>
<td>m</td>
<td>Spacing between center of test pile and center of reaction system</td>
</tr>
<tr>
<td>SPT</td>
<td></td>
<td>Standard penetration test</td>
</tr>
<tr>
<td>$u_{excess}$</td>
<td>kN/m$^2$</td>
<td>excess pore water pressure</td>
</tr>
<tr>
<td>$x_{max}$</td>
<td>m</td>
<td>Outer geometry dimension</td>
</tr>
<tr>
<td>$x_{min}$</td>
<td>m</td>
<td>Outer geometry dimension</td>
</tr>
<tr>
<td>$y_{max}$</td>
<td>m</td>
<td>Outer geometry dimension</td>
</tr>
<tr>
<td>$y_{min}$</td>
<td>m</td>
<td>Outer geometry dimension</td>
</tr>
<tr>
<td>$y_{ref}$</td>
<td>m</td>
<td>Reference depth</td>
</tr>
<tr>
<td>$\alpha$</td>
<td></td>
<td>Adhesion factor</td>
</tr>
<tr>
<td>Symbol</td>
<td>Units</td>
<td>Meaning</td>
</tr>
<tr>
<td>--------</td>
<td>-------</td>
<td>---------</td>
</tr>
<tr>
<td>$\gamma_{\text{unsat}}$</td>
<td>kN/m$^3$</td>
<td>Unsaturated unit weight of soil</td>
</tr>
<tr>
<td>$\gamma_{\text{sat}}$</td>
<td>kN/m$^3$</td>
<td>Saturated unit weight of soil</td>
</tr>
<tr>
<td>$\gamma_w$</td>
<td>kN/m$^3$</td>
<td>Unit weight of water</td>
</tr>
<tr>
<td>$\delta$</td>
<td>m</td>
<td>Movement of pile head</td>
</tr>
<tr>
<td>$\delta(r)$</td>
<td>m</td>
<td>Ground movement at a distance $r$ from the center of footing</td>
</tr>
<tr>
<td>$\delta(r_0)$</td>
<td>m</td>
<td>Settlement of the rigid footing</td>
</tr>
<tr>
<td>$\varepsilon_1$</td>
<td></td>
<td>Vertical strain</td>
</tr>
<tr>
<td>$\rho$</td>
<td>m</td>
<td>True settlement of loaded pile</td>
</tr>
<tr>
<td>$\rho_m$</td>
<td>m</td>
<td>Measured settlement</td>
</tr>
<tr>
<td>$\sigma'$</td>
<td>kN/m$^2$</td>
<td>Vector notation of effective normal stress</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>kN/m$^2$</td>
<td>Confining pressure in a triaxial test</td>
</tr>
<tr>
<td>$\sigma_h$</td>
<td>kN/m$^2$</td>
<td>Horizontal stress</td>
</tr>
<tr>
<td>$\sigma_n$</td>
<td>kN/m$^2$</td>
<td>Normal stress of soil</td>
</tr>
<tr>
<td>$\sigma_w$</td>
<td>kN/m$^2$</td>
<td>Pore pressure</td>
</tr>
<tr>
<td>$\varepsilon_{ij}$</td>
<td></td>
<td>Cartesian normal strain component</td>
</tr>
<tr>
<td>$\gamma_{ij}$</td>
<td></td>
<td>Cartesian shear strain component</td>
</tr>
<tr>
<td>$\tau$</td>
<td>kN/m$^2$</td>
<td>Shear strength of soil</td>
</tr>
<tr>
<td>$\nu$</td>
<td></td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>$\nu_u$</td>
<td></td>
<td>Poisson’s ratio for undrained</td>
</tr>
<tr>
<td>$\nu_{ur}$</td>
<td></td>
<td>Poisson’s ratio for unloading and reloading</td>
</tr>
<tr>
<td>$\phi$</td>
<td>o</td>
<td>Internal friction angle</td>
</tr>
<tr>
<td>$\phi'/\phi_{\text{soil}}$</td>
<td>o</td>
<td>Effective friction angle of the soil</td>
</tr>
<tr>
<td>$\psi$</td>
<td>o</td>
<td>Dilatancy angle</td>
</tr>
</tbody>
</table>
CHAPTER 1
INTRODUCTION

1.1 Objectives

Pile load test is a fundamental part of pile foundation design. It can afford an effective way to check on the uncertainties in soil parameter measurement and design assumptions that occurs in the design and construction of piles. A variety of test methods are to be found in the industry, ranging from full-scale static tests, with application of load and monitoring of pile deformation, to the measurement of associated properties of pile-soil system, for example in low-strain integrity tests. The list includes static load tests, statnamic and pseudo-static tests, Osterberg-cell test, dynamic test (in which a pile is struck by a falling hammer), and integrity tests (which basically use wave propagation and acoustic impedance measurement techniques to look only at structural continuity and implied section variation). The most essential information provided by pile test includes:

1) The ultimate load capacity of a single pile;
2) The load transfer behavior of a pile;
3) The load-settlement behavior of a pile;
4) The structural integrity of a pile as constructed.

Such information may be used as a means of verification of design assumptions as well as obtaining design data on pile performance which may allow for a more effective and confident design of the piles in a particular site.
Although many pile tests have been constructed in all kinds of engineering projects, it is hard to say that the results can afford reliable and unequivocal information which can be applied directly to the design process. We need to be very careful in the following aspects during the interpretation of pile test. These include:

1) Whether the test load on the pile is applied the same manner as the structure will load the prototype piles;
2) Whether the test set-up induces inappropriate stress changes in the ground or cause inaccuracies in the measurements of settlement;
3) Whether other factors exist that may have other side-effects on the result.

Unless all these aspects are considered and excluded from the measurement, a reasonable interpretation of the pile test would be difficult. Of course, in reality, it is highly unlikely that any one test procedure can simultaneously meet all of the above requirements of the designer. However, with the development of the numerical methods and the improvement of the performance of computers, the extent to which these tests can satisfy the above requirements of the designer can be extended by simulating the pile loading test in a numerical model and analyzing the results in combination with the field test data.

In this thesis, the finite element method (FEM) was used to carry out the research. This method has the advantage over traditional analysis techniques as more realistic test condition can be taken into account and displacements and stresses within the soil body and pile are coupled, thus more realistic pile-soil interaction behaviour can be represented with more realistic assumptions. The commercial finite element code
PLAXIS and PLAXIS 3D Foundation were used for the numerical simulation of pile load test that will be studied in the following.

1.2 Scope of Study

Due to the limitation of the time and length of the thesis, only some particular interest which is associated with the conventional static load test and Osterberg-cell test were studied. Different reaction systems for the static pile load test are analyzed to study the effect of reaction system on the test results; O-cell test is compared with static pile load test and equivalency and discrepancy of the test results between the two types of pile load test are demonstrated.

To fulfill the objectives of the research, the overall project is divided into six major tasks as follows:

Task 1. Literature review—The set-up of static pile load tests with different reaction system such as kentledge and reaction piles are described. The common recommendations of the spacing between the reaction system and the test pile are introduced and the study on the influence of the reaction system on the load-movement behaviour is reviewed. Besides, the principles of O-cell test are illustrated and some research work both in numerical and practical aspects on the O-cell test is highlighted.

Task 2. FEM study on the influence of the spacing between test pile and reaction system on the settlement of test pile; influence of geometric factors such as pile diameter, D, length/diameter ratio, L/D, or kentledge width B on the settlement of test
pile; the influence of soil parameters such as stiffness ratio $E_{\text{pile}}/E_{\text{soil}}$ on the settlement of test pile.

Task 3. FEM study to verify the assumptions that the shaft resistance-movement curve for upward movement of the pile in O-cell test is the same as the downward side-movement component of a conventional head-down test, while the end bearing load-movement curve obtained from an O-cell test is the same as the end bearing-load movement component curve of a conventional head-down test. The method to construct the equivalent top-loaded load-movement curve from the results of the O-cell test is discussed given that the pile is considered rigid and flexible respectively. Differences between the conventional test and O-cell test were analyzed and discussed.

Task 4. Case history of the O-cell test in Gopeng Street Project is re-analyzed and the numerical results are compared with the reported field measurements. They are used to illustrate the validity of the O-cell test as a good substitute for the conventional test. The advantage of the FEM simulation to the interpretation of the test result is also demonstrated.

Task 5. Case history of Harbour of Thessaloniki project is re-calculated with 3D FEM model to further verify the influence factors of reaction piles in practice.

Task 6. Case history of the Kentledge static load test in NTUC is studied to illustrate the discrepancy of the settlement, shaft and end bearing resistance with or without considering the influence of the Kentledge weight.
CHAPTER 2
LITERATURE REVIEW

2.1 Review of Pile Load Test

A number of forms of pile load test have been used in practice. Some methods such as static loading test and dynamic test have been a routine in geotechnical engineering for many years, while Osterberg cell test and statnamic test have been developed for less than twenty years. This thesis concentrates on the static loading test and Osterberg cell test as they are widely used in geotechnical area in Singapore and the test procedures and results can be modeled by finite element analysis method, so that the actual soil-pile relationships of ultimate capacity, distribution between shaft resistance and end bearing, load settlement response of the particular characteristics assumed in the design can be re-analyzed and verified by the finite element model.

Static load test is the most basic test and involves the application of vertical load directly to the pile head. Loading is generally either by discrete increases of load over a series of intervals of time (Maintained Load test and Quick Load test) or, alternatively, in such a manner that the pile head is pushed downward at a constant rate (Constant Rate Penetration test). Test procedures have been developed and defined by various codes, for example, ASTM D1143 and CIRIA ISBN 086017 1361. The test may take several forms according to the different reaction systems applied for the loading. Figs. 2.1, 2.2 and 2.3 illustrate kentledge reaction system, tension pile reaction system and ground anchor reaction system respectively that are commonly used in practice. Load-settlement curve is constructed simply by plotting the loads applied onto the pile head vs. the pile head displacement. The static load test is generally
regarded as the definitive test and the one against which other types of test are compared.

The Osterberg Cell (O-cell) method was developed by Osterberg (1989) while a similar test has been developed in Japan (Fujioka and Yamada, 1994). This method incorporates a sacrificial hydraulic jack (Osterberg Cell) placed at or near the toe of the pile, which divide the test pile into the upper and lower parts, see Fig.2.4. The test consists of applying load increments to both parts of pile by means of incrementally increasing the pressure in the jack, which causes the O-cell to expand, pushing the upper part upward and lower part downward simultaneously. The measurements recorded are the O-cell pressure (the load), the upward and downward movements, and the expansion of the O-cell. The O-cell load versus the upward movement of the O-cell top is the load-movement curve of the pile shaft. The O-cell load versus the downward movement of the O-cell base is the load-movement curve of the pile toe. This separate information on the load-movement behaviors of the shaft and toe is not obtainable in a conventional static loading test.

2.2 Reaction System and Static Load Test

2.2.1 Recommended Distance of Reaction System for Static Load Test

The ideal static load test of pile is one where the pile is subjected to “pure” vertical loading while no reaction system is necessary. It best simulates the way in which a structural building load is applied to the pile. However, this ideal test cannot usually be achieved in practice and loading the pile incrementally always leads to the change of load of reaction system. In the kentledge system, the deadweight of the kentledge loads
the soil around the pile at the beginning of the pile load test, and then unloads the soil with the increasing loading on the test pile head. While in the application of tension pile reaction system, the upward loads of the anchor piles cause an upward movement of the surrounding soil. Both of the service conditions of the pile load test cause the different stress changes in the soil surrounding the test pile with that in the ideal static load test. Hence, the interaction between the test pile and reaction system may cause errors in settlement and bearing capacity measurement of test pile.

To minimize the errors caused by the interaction of reaction system, recommendations are made regarding the minimum distance of reaction system to the test pile in all kinds of standards and papers. For example, ASTM (1987) suggests the clear distance between the test pile and the reaction pile(s) or cribbing shall be at least five times the butt diameter or diagonal dimension of the test pile, but not less than 2.5m; it also notes that factors such as type and depth of reaction, soil conditions, and magnitude of loads should be considered. When testing large diameter drilled shafts, the practicality of above mentioned spacing should be considered and the standard modified as warranted.

The minimum distance of 1.3m between the nearest edge of the crib supporting the kentledge stack to the surface is regulated, while a distance of at least three test pile shaft diameters from the test pile, centre to centre, and in no case less than 2m is recommended in BS 8004:1986, Singapore Standard CP4-2003 and Tomlinson (1994).

Weltman (1980) considers a distance from the face of the test pile of 1.0m should be appropriate in the kentledge reaction system while in tension pile reaction system, at
least 8d (diameter of the pile) would be entailed, whereas 3 to 4d is employed and a lower limit of 2.0m is recommended in practice.

Some other recommendations are collected and listed in Table.2.1. It is noted that the significant interaction between test pile and reaction system within 3 times diameters of test pile is a common sense. Also, it seems that the interaction between reaction pile system and test pile is greater than that of kentledge reaction system. Finally, the extent of the interaction effects may change due to the soil condition, load level, pile dimensions etc., which requires the geotechnical engineer to make proper adjustment to the available spacing according to the field circumstances that reduce the influence of interaction to an acceptable degree.

Table. 2.1 Recommended Spacing between Test Pile and Reaction System

<table>
<thead>
<tr>
<th>Reference</th>
<th>Recommended spacing for kentledge reaction system</th>
<th>Recommended spacing for tension pile reaction system</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM(1987)</td>
<td>Clear distance $\geq 5d$ or $\geq 2.5m$</td>
<td>Clear distance $\geq 5d$ or $\geq 2.5m$</td>
</tr>
<tr>
<td>ASCE(1976)</td>
<td>$\geq 8d$</td>
<td></td>
</tr>
<tr>
<td>BS8004:1986</td>
<td>$\geq 1.3m$</td>
<td>$\geq 3$ or $4d$ and $\geq 2.0m$</td>
</tr>
<tr>
<td>ICE(1978)</td>
<td>$\geq 1.3m$</td>
<td>$\geq 3$ or $4d$ and $\geq 2.0m$</td>
</tr>
<tr>
<td>NYSDOT(1977)</td>
<td>$\geq 3m$ or $\geq 10d$</td>
<td></td>
</tr>
<tr>
<td>Weltman (1980)</td>
<td>Clear distance $\geq 1m$</td>
<td>$\geq 8d$</td>
</tr>
<tr>
<td>Fleming, et al. (1992)</td>
<td>$\geq 3$–$4d$</td>
<td></td>
</tr>
<tr>
<td>Poulos and Mattes (1975)</td>
<td></td>
<td>$\geq 10d$ for long pile $\geq 5d$ for short pile</td>
</tr>
<tr>
<td>Nair (1967)</td>
<td></td>
<td>$\geq 15d$</td>
</tr>
</tbody>
</table>

Note: ASCE - American Society of Civil Engineers
ASTM - American Society for Testing and Materials
2.2.2 Interaction Effect of Reaction System on the Results of Static load Test

For the static load test, the influence of reaction system on the ultimate capacity and load-settlement behaviour of the test pile is reported in many papers.

Weltman (1980) indicated the cribbage pads should be spaced away enough from the test pile to avoid the interaction. Even at a recommended minimum spacing of 1.0m, some interaction would occur. For the tension pile reaction system, he indicated that the settlement of an individual pile could be underestimated by more than 20% depending on the soil conditions in the cases that minimum spacing of 3 to 4d or a lower limit of 2.0m is employed.

Weele (1993) illustrated the interaction effect of both kentledge and tension pile reaction systems in two pile load tests. Fig. 2.5 presents the site data while Fig. 2.6 shows the result of two load tests on the same pile. Load test 1 was performed with 6 neighbouring piles acting as anchor piles, while test 2 was performed using 200 tones of kentledge, supported by the same neighbouring piles. The test with kentledge gave a failure load of 2300 kN, whereas the test with the anchor piles gave only 1350 kN. The observed difference is determined by pile size, soil conditions, pile distances, failure load, etc. The test indicated that there is thus no fixed relation between both, but tests using the weight of the soil, surrounding the pile, will always render a lower ultimate capacity and a “softer” load/settlement behavior than the test using dead weight.
Latotzke et al. (1997) carried out a series of centrifuge model tests to prove that a significant difference exists between the load-settlement behaviour observed by modeling the in-situ procedure and the load-settlement behaviour of the single pile without interaction effects. Some results are shown in Fig. 2.7 and 2.8, indicating that the bearing capacity of the test pile observed from the combined pile system is higher than the bearing capacity observed from the single pile system concerning equal settlement; the total bearing capacity of the test is highly influenced by the reaction piles concerning small settlement and for larger settlements the shaft resistance is reduced by the influence of the reaction piles which leads to a smaller influence on the total bearing capacity. By plotting the influence factors \( f, f_s \) and \( f_T \) versus dimensionless settlement \( s/D \) in Fig.2.9, it is obvious that the measured bearing capacity of the combined pile system is nearly 70% larger than that of the uninfluenced single pile up to the settlement of \( s/D=0.1 \), which is relevant for practical design.

where

\[
f = \frac{Q_{CPS} - Q_{SPS}}{Q_{SPS}} \quad (2.1)
\]

\[
f_s = \frac{Q_{S,CPS} - Q_{S,SPS}}{Q_{SPS}} \quad (2.2)
\]

\[
f_T = \frac{Q_{T,CPS} - Q_{T,SPS}}{Q_{SPS}} \quad (2.3)
\]

where:

\( Q = \) total load, \( Q_S = \) shaft resistance, \( Q_T = \) tip resistance

SPS= single pile system \hspace{1cm} CPS = combined pile system

Lo (1997) carried out a series of field pullout tests on tension piles to investigate the effects of ground reaction stresses on the pile performance. The results suggested that
the interaction between the knowledge support and test pile led to an over-prediction of the ultimate uplift capacity of the pile up to about 10–20% and an underestimate of the pile head displacement. These field tests were consistent with the theoretical results obtained from non-linear finite element analysis assuming the soil to be uniform sand exhibiting an ideal elastic-plastic behaviour, see Fig.2.10.

Some theoretical analyses have been made with different numerical methods. The effects of interaction between reaction piles and the test pile have been examined theoretically with elastic method by Poulos and Davis (1980). In this method, soil is considered as a continuum and the classical theory of elasticity is applied. The pile is divided into a number of uniformly loaded elements, and a solution is obtained by imposing adjacent soil for each element of the pile. The displacements of the pile are obtained by considering the compressibility of the pile under axial loading. By using Mindlin's equations for the displacements within a soil mass caused by loading within the mass, the soil displacements are obtained.

They used this method in the analysis of static pile load test with different reaction systems, such as reaction pile system and ground anchor system. With this method of load application, the upward loads on the anchor piles cause an upward movement of the test pile because of interaction. Therefore, the measured settlement is equal to the true settlement of the ideal axially-loaded pile, which is the calculated settlement without considering the interaction of reaction system using this method, minus the displacement caused by the reaction system. As a result, the measured settlement will be less than the true settlement and the pile head stiffness will be overestimated as well. To minimize the error, a correction factor, $F_c$, is defined as:
Values of $F_c$ for various cases are plotted in Figs. 2.11 and 2.12. The case of a floating pile in a deep soil layer is considered in Fig. 2.11. It may be seen that in the range of spacings between the test and reaction piles commonly used (2.5 to 4 diameters), $F_c$ may be 2 or even greater. The error becomes more severe for stiffer, more slender piles. Fig. 2.12 shows values of $F_c$ for end-bearing piles resting on a rigid stratum. In this case, the interaction is generally much less, and consequently, large values of $F_c$ do not occur at normal spacing unless the piles are relatively slender and compressible. Both cases suggest that the usual spacing of about three diameters may result in significant under-measurement of the settlement of the test pile. Increasing the spacing to at least five diameters would appear most desirable, especially for long piles in deep, soft deposits.

Zheng (1999) made a nonlinear analysis taking into account the small strain stiffness variation for soil on the influence of the rectangular-shaped kentledge cribbage on the test pile. Assuming the influence of the kentledge is expected to lie between the influence of a circular footing with a diameter the same as the width of the cribbage and that of a strip footing with the same width, she studied the parameters such as width of cribbage, $B$, undrained shear strength of soil, $C_u$. The results are presented in Figs 2.13 and 2.14 in the form of normalized displacement of the ground surface, $\delta(r)/\delta(r_0)$ versus the normalized distance $r/B$, in which, $r$ is the distance from the center of

\[ F_c = \frac{\rho}{\rho_m} \]  

where \( \rho = \) True settlement of loaded pile  
\( \rho_m = \) Measured settlement
footing; B is the diameter of circular footing or the width of strip footing; \( \delta(r) \) is the ground movement at a distance \( r \) from the center of footing; and \( \delta(r_0) \) is the settlement of the rigid footing. Fig.2.13 indicates that by keeping the area of cribbage unchanged and changing the L/B ratio, the geometry of the kentledge cribbage had no influence on the settlement of the test pile. Fig.2.14 shows that lower \( C_u \) value causes more non-linearity of soil. Furthermore, the normalized ground settlement reduces sharply with lower undrained shear strength for soil. However, due to the limitations of plane strain analysis, her calculations didn’t consider the interaction between the test pile and kentledge.

With the same method, Zheng (1999) has analyzed pile load test using two reaction piles under working load in non-homogenous London clay with soil stiffness proportional to depth, in which parametric studies were conducted to illustrate the influence of the pile diameter \( D \) and the L/D (L is the length of the pile) etc., on the interaction factor ratio \( \beta \) (the ratio of the interaction factor \( \alpha_2 \) for two piles at a spacing of ‘2S’ over the interaction factor \( \alpha_1 \) for two piles at a spacing of ‘S’) in different soils. Fig. 2.15 illustrated that for different diameters of pile with the same L/D ratio, \( D \) does not affect the interaction factor ratio \( \beta \). It is also founded from Fig. 2.15 that the value of the interaction factor ratio \( \beta \) increases with L/D ratio, and decreases with S/D increasing. By comparing the value of \( \beta \) under the different \( C_u \), Fig.2.16 showed that \( C_u \) has a negligible influence on the interaction factor ratio \( \beta \). Besides, the author also noted that the results of quasi-nonlinear analysis for the interaction factor ratio \( \beta \) are close to those of the linear elastic analysis at working load.
2.3 Comparison of O-cell Test with Static Load Test

It is well known that conventional static load test has inherent disadvantages. The influence of reaction system may be reduced by increasing the spacing between test pile and reaction pile or kentledge; however, it is not always achieved when the working space is restrained. Besides, the interpretation of the data obtained from conventional tests is not straightforward as it is not easy to separate the shaft resistance from the end bearing capacity.

On the other hand, O-cell load test makes use of shaft resistance above the top of the O-cell as reaction to load the downward base of O-cell, thus avoiding the influence of reaction system in the conventional static load test. At the same time, shaft resistance and end bearing components of the total bearing capacity of test pile are separated automatically. However, the loading mechanism of O-cell load test is not like that of conventional head-down test, which coincides with the real loading status of foundation that loading is from top downward. Besides, as an O-cell test usually reaches the ultimate load in only one of the two resistance components, it is always needed to extrapolate the load curve data for the other component. Although the validity of the O-cell test has been confirmed, to what extent that the O-cell test can represent the conventional load test is still a debatable topic.

Osterberg (1998) indicates that the upward movement-shaft resistance curve and the downward movement-end bearing curve of O-cell load test can be used to reconstruct the head-down equivalent curve of conventional load test on the basis of three assumptions:
1) The shaft resistance-movement curve for upward movement of the pile is the same as the downward shaft resistance-movement component of a conventional head-down test.

2) The end bearing load-movement curve obtained from an O-cell test is the same as the end bearing-load movement component curve of a conventional head-down test.

3) The pile is considered rigid. This is coming from the experience that for bored concrete piles the compression of the pile is typically 1-3mm at ultimate load.

To verify the validity of assumptions 1 and 2, a series of tests have been carried out in Japan. One of the tests is made up of a pile with 1.2m in diameter and 26.5m in length. The hole was bored using drilling mud and the concrete was placed under drilling mud with a tremie. Fig.2.17 shows the comparison of the movement-end bearing curve obtained from the O-cell test with that obtained from the head-down test. Fig.2.18 illustrate the comparison between the measured head-down test data and calculated data by load transfer analysis using the shaft resistance obtained by O-cell reading. The close agreement of these curves indicates that assumption 1 is quite reasonable.

In another test, the pile was first tested by pushing up from the bottom with the preinstalled O-cell and then pushing down from the top with a jack on the top of the pile while the O-cell was depressurized at the time so that there is no end bearing. The result showed in Fig.2.19 provides the evidence of validity of the O-cell test being essentially the same as a conventional head down test in shaft resistance.
Poulos et al. (2000) made a numerical analysis with the commercial program FLAC on a hypothetical case of a pile in medium sand bearing on a denser sand layer. The results of an “ideal” static compression test are shown in Fig.2.20 together with the results of the Osterberg cell test. It is concluded that the results are overall comparable, with the O-cell test giving a slightly stronger response under small settlement and smaller ultimate and base capacities thereafter. They also pointed out that there is interaction between the base and the shaft during the O-cell test, each will tend to be larger than “real” movement so that the apparent shaft and base stiffness will tend to be larger than the real value.

Fellenius et al. (1999) performed a FEM analysis on an O-cell test of 28-m-deep barrette in Manila, Philippines. To respond to the mentioned suggestion that the O-cell test would be fundamentally different from a conventional head-down static loading test, a conventional static loading test was simulated in a repeated FEM computation. Fig.2.21 presents the distribution of axial load in the barrette for the two types of test. The left of the two head-down curves is for the case of a maximum load applied to the barrette head equal to twice the net O-cell test load during the initial test. The right of the two curves is for the case of equal base movement, which required a slightly larger total load to be imposed at the barrette head. The approximate tangent of the two curves at the counterpart elevation showed the same amount of shaft resistance developed along the pile shaft. The same amount of end bearing is evidenced at the barrette base. Fig.2.22 presents the unit shaft resistance distribution (shaft resistance) for the barrette as calculated for both types of tests. The plot was displayed in such a mode that one curve of unit shaft resistance versus depth looks like the mirror image of another, which indicates very little difference between the computed unit side-shear
values for the two types of tests. Fig.2.23 shows the recorded base and shaft O-cell curves together with the equivalent head-down curves for rigid and non-rigid considerations of the pile. When comparing the rigid and non-rigid curves, the importance of including the elastic shortening of the pile is obvious.

2.4 Finite Element Analysis

2.4.1 Review of Theoretical Method

The load-settlement behavior and ultimate load capacity of the pile are two main issues that are concerned about when conducting a pile load test. The relevant theoretical analysis of static pile load test is based on analysis of the single pile under the axial compression.

With the advent of computers, more sophisticated methods of analysis have been developed to predict the settlement and load distribution in a single pile. In general, there are three broad categories:

1) Load-Transfer Method. This method was first developed by Seed and Reese (1957), which used soil data measured from field tests on instrumented piles and laboratory tests on model piles to build the relationships between pile resistance and pile movement at various points along the pile. Because it is inherently assumed that the movement of the pile at any point is related only to the shear stress at that point and is independent of the stresses elsewhere on the pile, no proper account is taken of the continuity of the soil mass. Besides, precise load-transfer curve needs more instrumentations than for a normal pile load test.
2) Elastic method. Elastic-based analyses have been employed by several researchers, for example, Nair (1967), Poulos and Davis (1968), Randolph and Wroth (1978). In this method, the piles are divided into a number of uniformly-loaded elements and the soil acts as elastic solid, a solution is obtained by imposing compatibility between the displacements of the pile and the adjacent soil for each segment of the pile. The displacements of the pile are calculated by considering the compressibility of the pile under axial loading while the soil displacements are usually obtained by using Mindlin’s equations. However, due to the limitation of the linear elastic soil model, pile-soil interaction in pile load test is always overestimated.

3) Numerical Method. Of the various numerical methods, the finite element technique allows more variables to be considered in the problem. Ellison et al. (1971) have considered a multilinear soil stress-strain curve and have introduced special joint elements at the pile interface to allow for slip. Other investigators include Desai (1974) etc. The method involves discretizing of the pile and soil domains into a finite number of elements. Stiffness equations are formulated for each element and assembled together to give the global system. The appropriate constitutive models are selected to simulate the stress-strain behavior of soil so that soil inhomogeneity and nonlinearity can be studied in a rigorous manner. With the development of high performance PC, some powerful FEM programs such as CRISP and PLAXIS have been widely used in research, which made it possible that more factors such as 3D effects can be taken into consideration so that more realistic situations can be simulated.
2.4.2 Introduction to PLAXIS and PLAXIS 3D Foundation

2.4.2.1 General

PLAXIS v.8 and PLAXIS 3D Foundation are two finite element codes used for the numerical simulation of pile load test in this thesis.

PLAXIS is a 2D finite element package for the analysis of deformation and stress of the soil and soil-structure interaction problems. Geotechnical applications require constitutive models for the realistic simulation of the non-linear and time dependent behaviour of soils. PLAXIS has the required features to deal with numerous problems encountered in most geotechnical structures.

PLAXIS 3D Foundation is a family member of PLAXIS, which is a special purpose three-dimensional finite element computer program used to perform deformation analyses for various types of foundations in soil and rock. The program allows for a fully automatic generation of 2D and 3D finite element meshes, which enables users to quickly generate a true three-dimensional finite element mesh based on a composition of horizontal cross sections at different vertical levels.

2.4.2.2 Model

In PLAXIS 8.0, plane strain model can be used for structures with an almost uniform cross section, corresponding stress state and loading scheme over a certain length perpendicular to the cross section. Displacements perpendicular to the cross section are assumed to be zero. Axisymmetric model can be used for circular structures with
uniform radial cross section and loading scheme around the central axis, where deformation and stress state are assumed to be identical in any radial direction. To analyze the problem of pile, the axisymmetric model should be selected, which results in a two dimensional finite element model with only two translational degrees of freedom at each node (i.e. x- and y- direction).

In PLAXIS 3D Foundation, the generation of a 3D finite element model begins with the creation of a geometry model. A geometry model is a composition of bore holes and horizontal work planes. The work planes are used to define geometry lines and structures contour lines along the elevation level. The bore holes are used to define the local soil stratigraphy, ground surface level and pore pressure distribution. From the geometry model, a 2D mesh is generated first, after which an extension into the third dimension (the y-direction) can be made. PLAXIS 3D Foundation automatically generates this 3D mesh, taking into account the information from the work planes and the bore holes. Thus the full 3D geometry model including all objects appearing in any work plane at any construction stage has been defined.

PLAXIS 3D Foundation has various special elements to model all kinds of structures, such as beam, floor, and wall elements. However, no special type of element is applied to model the pile. Representing the pile with 3D solid element limits the numbers of the piles that can be modeled due to the memory capacity of the PC.

2.4.2.3 Elements

In PLAXIS 8, 6-node or 15-node triangular elements are available. six-node triangle provides a second order interpolation for displacements. The element stiffness matrix is evaluated by numerical integration using three Gauss points. For the 15-node
triangle, the order of interpolation is four and numerical integration involves twelve Gauss points.

In PLAXIS 3D Foundation, the basic soil elements of a 3D finite element mesh are the 15-node wedge elements. These elements are generated from the 6-node triangular elements as generated in the 2D mesh. Due to the presence of non-horizontal soil layers, some 15-node wedge elements may degenerate to 13-node pyramid elements or even to 10-node tetrahedral elements. The 15-node wedge element is composed of 6-node triangles in horizontal direction and 8-node quadrilaterals in vertical direction. The accuracy of the 15-node wedge element and the compatible structural elements are comparable with the 6-node triangular elements in a 2D PLAXIS analysis. Higher order element types, for example comparable with the 15-node triangle in a 2D analysis, are not considered for a 3D Foundation analysis because this will lead to large memory consumption and unacceptable calculation times.

The floor element which is applied in this thesis is an exclusive element in PLAXIS 3D Foundation compared with PLAXIS 8. Floors are structural objects used to model thin horizontal (two-dimensional) structures in the ground with a significant flexural rigidity (bending stiffness). It is composed of 6-node triangular plate elements with six degrees of freedom per node: Three translational degrees of freedom and three rotational degrees. Element stiffness matrices and plate forces are numerically integrated from the $2 \times 3$ Gaussian integration points (stress points). The plate elements are based on Mindlin’s plate theory.
2.4.2.4 Interfaces

Interfaces are used when modeling soil structure interaction. Interfaces will be required to simulate the finite frictional resistance between the structure such as pile and adjacent soil. It allows relative displacement and separation between the structure and soil mass.

When using 6-node elements for soil, the corresponding interface elements are defined by three pairs of nodes, whereas for 15-node soil elements the corresponding interface elements are defined by five pairs of nodes.

The stiffness matrix for interface elements is obtained using Newton-Cotes integration points. The position of these integration points coincides with the position of the node pairs. The 6-node interface elements use a 3-point Newton-Cotes integration, whereas the 10-node interface elements use 5-point Newton-Cotes integration.

The basic property of an interface element is the associated material data set for soil and interfaces. When interface element models the interaction between a pile and the soil, which is intermediate between smooth and fully rough. The roughness of the interaction is modeled by choosing a suitable value for the strength reduction factor in the interface ($R_{inter}$). This factor relates the interface strength (structure surface friction and adhesion) to the soil strength (friction angle and cohesion). An elastic-plastic model is used to describe the behaviour of interfaces for the modeling of soil-structure interaction. The Coulomb criterion is used to distinguish between elastic behaviour, where small displacements can occur within the interface, and plastic interface behaviour when permanent slip may occur.
For the interface to remain elastic the shear stress $\tau$ is given by:

$$|\tau| < \sigma_n \tan \phi_i + c_i$$  \hspace{1cm} (2.5)

and for plastic behaviour $\tau$ is given by:

$$|\tau| = \sigma_n \tan \phi_i + c_i$$  \hspace{1cm} (2.6)

where $\phi_i$ and $c_i$ are the friction angle and cohesion (adhesion) of the interface, $\sigma_n$ is the normal stress of the soil. The strength properties of interfaces are linked to the strength properties of a soil layer. Each data set has an associated strength reduction factor for interface ($R_{inter}$). The interface properties are calculated from the soil properties in the associated data set and the strength reduction factor by applying the following rules:

$$c_i = R_{inter} c_{soil} \leq c_{soil}$$  \hspace{1cm} (2.7)

$$\tan \phi_i = R_{inter} \tan \phi_{soil} \leq \tan \phi_{soil}$$  \hspace{1cm} (2.8)

### 2.4.2.5 Linear Elastic Model

This model represents Hooke’s law of isotropic linear elasticity. The model involves two elastic stiffness parameters, i.e. Young’s modulus, $E$, and Poisson’s ratio, $\nu$. The linear elastic model is seldom used to simulate soil behaviour. It is primarily used for stiff massive structural systems install in the soil, such as the test pile in this thesis.

### 2.4.2.6 Mohr-Coulomb Model

This well known model is usually used as a first approximation of soil behaviour. Due to its simplicity, it is highly popular and gives reasonable results. The model involves five parameters, i.e. Young’s modulus, $E$, Poisson’s ratio, $\nu$, cohesion, $c$, internal friction angle, $\phi$, and dilatancy angle, $\psi$. 
In real soils, the stiffness depends significantly on the stress level, which means that the stiffness generally increases with depth. The advanced M-C model in PLAXIS provides an option to account for the increase of the stiffness with depth. The $E_{\text{increment}}$ is the increase of the Young’s modulus per unit of depth (expressed in the unit of stress per unit depth). At the level given by the $y_{\text{ref}}$ parameter, the stiffness is equal to the reference Young’s modulus, $E_{\text{ref}}$. The actual value of Young’s modulus in the stress points is obtained by Eq.2.9.

$$E_{\text{actual}} = E_{\text{ref}} + (y_{\text{ref}} - y)E_{\text{increment}} \quad y < y_{\text{ref}} \quad (2.9)$$

$$c_{\text{actual}} = c_{\text{ref}} + (y_{\text{ref}} - y)c_{\text{increment}} \quad y < y_{\text{ref}} \quad (2.10)$$

However, during calculations a stiffness increasing with depth does not change as a function of the stress state. Similarly, the increase of the cohesion with depth is accounted for in the M-C model in PLAXIS, as in Eq.2.10.

### 2.4.2.7 Hardening Soil Model

The Hardening-Soil model is an advanced model developed by Schanz and Vermeer (1998) for simulating the behaviour of different types of soil, both soft soils and stiff soils. When subjected to primary deviatoric loading, soil shows a decreasing stiffness and simultaneously irreversible plastic strains develop. The observed relationship between the axial strain and the deviatoric stress can be well approximated by a hyperbola in the special case of a drained triaxial test. Such a relationship was first formulated by Kondner (1963) and later used in the well-known hyperbolic model (Duncan & Chang, 1970). The general three-dimensional extension and implementation in PLAXIS dated back to Vermeer and Brinkgreve (1995). The Hardening-Soil model has the following advantages of the others: Firstly by using the
theory of plasticity rather than the theory of elasticity; secondly by including soil dilatancy and thirdly by introducing a yield cap.

The model requires more complicated parameters, i.e. cohesion, c, internal friction angle, \( \varphi \), dilatancy angle, \( \psi \), power for stress-level dependency of stiffness, m, secant stiffness in standard drained triaxial test, \( E_{50}^{\text{ref}} \), tangent stiffness for primary oedometer loading, \( E_{\text{oed}}^{\text{ref}} \), unloading/reloading stiffness, \( E_{\text{ur}}^{\text{ref}} \), Poisson’s ratio for unloading-reloading, \( \nu_{\text{ur}} \), coefficient of lateral stress in normal consolidation, \( K_0^{\text{NC}} \) etc. The following is a summary of the most important assumptions and approaches.

A basic idea for the formulation of the Hardening-Soil model is the hyperbolic relationship between the vertical strain, \( \varepsilon_1 \) and the deviatoric stress, \( q \), in primary triaxial loading. Standard drained triaxial tests tend to yield curves that can be described by:

\[
\varepsilon_1 = \frac{1}{2E_{50}} \frac{q}{1 - q/q_a} \quad \text{for: } q < q_a \tag{2.11}
\]

where \( q_a \) is the asymptotic value of the shear strength. This relationship is plotted in Fig. 2.24. The parameter \( E_{50} \) is the confining stress dependent stiffness modulus for primary loading and is given by the equation:

\[
E_{50} = E_{50}^{\text{ref}} \left( \frac{c \cos \varphi - \sigma_3' \sin \varphi}{c \cos \varphi + p^{\text{ref}} \sin \varphi} \right)^m \tag{2.12}
\]

where \( E_{50}^{\text{ref}} \) is a reference stiffness modulus corresponding to the reference confining pressure \( p^{\text{ref}} \). In PLAXIS, a default setting \( p^{\text{ref}}=100 \) stress units is used. The actual stiffness depends on the minor principal stress, \( \sigma_3' \), which is the confining pressure in a triaxial test. The amount of stress dependency is given by the power m, which is
reported varying in the range 0.5<m<1.0 (Von Soos, 1990), according to the soil type from sand and silts to soft clay.

The ultimate deviatoric stress, $q_f$ and the quantity $q_a$ in Eq.2.13 are defined as:

\[ q_f = (c \cot \varphi - \sigma_3') \frac{2 \sin \varphi}{1 - \sin \varphi} \quad \text{and:} \quad q_a = \frac{q_f}{R_f} \tag{2.13} \]

The above relationship for $q_f$ is derived from the Mohr-Coulomb failure criterion, which involves the strength parameters $c$ and $\varphi$. As soon as $q=q_f$, the failure criterion is satisfied and perfectly plastic yielding occurs as described by the Mohr-Coulomb model. $R_f$ is the failure ratio, defined as the ratio between $q_f$ and $q_a$.

For unloading and reloading stress paths, another stress-dependent stiffness modulus is used:

\[ E_{ur} = E_{ur}^{ref} \left( \frac{c \cos \varphi - \sigma_3' \sin \varphi}{c \cos \varphi + p^{ref} \sin \varphi} \right)^m \tag{2.14} \]

where $E_{ur}^{ref}$ is the reference Young’s modulus for unloading and reloading, corresponding to the reference pressure $p^{ref}$. In many practical cases it is appropriate to set $E_{ur}^{ref}$ equal to $3E_{50}^{ref}$.

\textbf{2.4.2.8 Mesh Properties}

PLAXIS allows for a fully automatic generation of finite element mesh. The generation of the mesh is based on a robust triangulation procedure, which results in “unstructured” meshes. These meshes may look disorderly, but the numerical performance of such meshes may yield better results than for regular structured meshes.
The mesh generator requires a general meshing parameter which represents the average element size, $l_e$, computed based on the outer geometry dimensions $(x_{\text{min}}, x_{\text{max}}, y_{\text{min}}, y_{\text{max}})$ using the following relationship:

$$l_e = \sqrt{\frac{(x_{\text{max}} - x_{\text{min}})(y_{\text{max}} - y_{\text{min}})}{n_c}}$$

(2.15)

where $n_c = 25$ (very coarse mesh)

= 50 (coarse mesh)

= 100 (medium mesh)

= 200 (fine mesh)

= 400 (very fine mesh)

2.4.2.9 Staged Construction

PLAXIS allows for the option to change the geometrical configurations by activating and deactivating clusters or structural objects, which is convenient to simulate the installation of the pile as a material wish-in-place, for the purpose of the study in the following. Also the program allows an accurate and realistic simulation of the actual construction stages such as the loading changes of the reaction system during the process of pile loading. The material properties and pore pressure distribution can also be changed at each stage.

2.4.2.10 Undrained Analysis with Effective Parameters

PLAXIS allows specifying undrained behaviour in an effective stress analysis using effective model parameters. This can be achieved by transforming the inverted form of Hooke’s law in terms of the total stress rates and the undrained parameters $E_u$ and $\nu_u$ according to Terzaghi’s principle, that is stresses in the soil are divided into effective stresses, $\sigma'$ and pore pressures, $\sigma_w$.
\[ \sigma = \sigma' + \sigma_w \quad (2.16) \]

In which, \( \sigma \) is vector notation involving six different components:
\[
\sigma = (\sigma_{xx} \ \sigma_{yy} \ \sigma_{zz} \ \sigma_{xy} \ \sigma_{yz} \ \sigma_{zx})^T \quad (2.17)
\]

A further distinction is made between steady state pore stress, \( p_{\text{steady}} \), and excess pore stress, \( u_{\text{excess}} \):
\[
\sigma_w = u_{\text{steady}} + u_{\text{excess}} \quad (2.18)
\]

Steady state pore pressures are considered to be input data, i.e. generated on the basis of phreatic levels or groundwater flow. Excess pore pressures are generated during plastic calculations for the case of undrained material behavior. Undrained material behaviour and the corresponding calculation of excess pore pressures are described below. Since the time derivative of the steady state component equals zero, it follows:
\[
\dot{\sigma}_w = \dot{u}_{\text{excess}} \quad (2.19)
\]

Considering slightly compressible water, the rate of pore pressure is written as:
\[
\dot{\sigma}_w = \frac{K_w}{n} (\dot{\varepsilon}_{xx}^e + \dot{\varepsilon}_{yy}^e + \dot{\varepsilon}_{zz}^e) \quad (2.20)
\]

in which \( K_w \) is the bulk modulus of the water and \( n \) is the soil porosity. The inverted form of Hooke’s law may be written in terms of the total stress rates and the undrained parameters \( E_u \) and \( \nu_u \):

\[
\begin{bmatrix}
\dot{\varepsilon}_{xx}^e \\
\dot{\varepsilon}_{yy}^e \\
\dot{\varepsilon}_{zz}^e \\
\dot{\gamma}_{xy}^e \\
\dot{\gamma}_{yz}^e \\
\dot{\gamma}_{zx}^e \\
\end{bmatrix} = \frac{1}{E_u} 
\begin{bmatrix}
1 & -\nu_u & -\nu_u & 0 & 0 & 0 \\
-\nu_u & 1 & -\nu_u & 0 & 0 & 0 \\
-\nu_u & -\nu_u & 1 & 0 & 0 & 0 \\
0 & 0 & 0 & 2 + 2\nu_u & 0 & 0 \\
0 & 0 & 0 & 0 & 2 + 2\nu_u & 0 \\
0 & 0 & 0 & 0 & 0 & 2 + 2\nu_u \\
\end{bmatrix} 
\begin{bmatrix}
\varepsilon_{xx} \\
\varepsilon_{yy} \\
\varepsilon_{zz} \\
\gamma_{xy} \\
\gamma_{yz} \\
\gamma_{zx} \\
\end{bmatrix} 
\]

\[
E_u = 2G(1 + \nu_u) \quad (2.22)
\]
\[
\nu_u = \frac{\nu' + \mu(1 + \nu')}{1 + 2\mu(1 + \nu')}
\]

(2.23)

\[
\mu = \frac{1}{3n} \frac{K_u}{K'}
\]

(2.24)

\[
K' = \frac{E'}{3(1 - 2\nu')}
\]

(2.25)

Hence, the special option for undrained behaviour in PLAXIS is such that the effective parameters \(G\) and \(\nu\) are transferred into undrained parameters \(E_u\) and \(\nu_u\) according to Eqs. 2.16~2.18.

Fully incompressible behaviour is obtained for \(\nu_u = 0.5\). However, it leads to singularity of the stiffness matrix. In fact, water is not fully incompressible although a realistic bulk modulus for water is very large. PLAXIS has taken a default \(\nu_u\) as 0.495 to avoid the numerical problems caused by an extremely low compressibility.

This special option to model undrained material behaviour on the basis of effective model parameters is available for all material models in the PLAXIS program. However, for such a project that in situ tests and laboratory test may have been performed to obtain undrained soil parameters, PLAXIS offers the option of an undrained analysis with direct input of the undrained shear strength \((c_u\) or \(s_u\)) and \(\phi=\phi_u=0^\circ\) in M-C model and H-S model.
Fig. 2.1 Schematic Set-Up for Static Pile Loading Test Using Kentledge
Fig. 2.2       Schematic Set-Up for Static Pile Loading Test Using Anchored Reaction Piles
Fig. 2.3 Schematic Set-Up for Static Pile Loading Test Using Ground Anchor
Fig. 2.4  Schematic Set-Up for Osterberg-Cell Test
Fig. 2.5  Plan with Location of CPT and 6 Anchor-piles (Weele, 1993)

Fig. 2.6  Result of 2 Load Tests on the Same Pile (Weele, 1993)
Fig 2.7  Comparison of Total Load, Skin Friction and Tip Resistance (Latotzke et al., 1997)

Fig 2.8  Comparison of Skin Friction with Settlement of the Test Piles (Latotzke et al., 1997)
Fig 2.9  Development of the Influence Factors with Settlements (Latotzke et al., 1997)

Fig 2.10  Example of Influence of Kentledge on Pile Test in Sand (Lo, 1997)
Fig. 2.11  Correction Factor $F_c$ for Floating Pile in a Deep Layer Jacked against Two Reaction Piles (Poulos and Davis, 1980)

Fig. 2.12  Correction Factor $F_c$ for End-bearing Pile on Rigid Stratum Jacked against Two Reaction Piles (Poulos and Davis, 1980)
Fig. 2.13  Comparison of Circular Footing and Strip Footing, When \( B=1 \text{m}, 2 \text{m} \) and 2.5m (Zheng, 1999)

Fig. 2.14  Comparison of Circular Footing and Strip Footing with Different \( C_u \) Values (Zheng, 1999)
Fig. 2.15 Interaction Factor Ratio $\beta$ for London Clay
($C'_{u} = 200$kN/m$^2$, non-homogeneous soil)
(Zheng, 1999)
Fig. 2.16 Interaction Factor Ratio $\beta$ for London Clay
(Upper: $C_u = 100\text{kN/m}^2$, non-homogeneous soil
Lower: $C_u = 50\text{kN/m2}$, non-homogeneous soil)
(Zheng, 1999)
Fig. 2.17  Comparison of the Deflection-end Bearing Curve of O-cell and Top Down Test (Osterberg, 1998)

Fig. 2.18  Comparison of the Load-Movement Curve of Measured and Calculated (Osterberg, 1998)
Fig. 2.19  Comparison of the Shaft Resistance Value (Osterberg, 1998)
Fig. 2.20  Theoretical Comparison between Ideal Tests and O-cell Test for Pile in Sand (Poulos et al., 2000)
Fig. 2.21 Vertical Load versus Depth for O-cell and Head test (Fellenius et al., 1999)

Fig. 2.22 Unit Side Shear versus Depth for O-cell and Head Test (Fellenius et al., 1999)
Fig. 2.23  Load-Movement for Equivalent Head-Down Test (Fellenius et al. 1999)

Fig. 2.24  Hyperbolic Stress-strain Relations in Primary Loading in Standard Drained Triaxial Test
CHAPTER 3

FEM STUDY ON EFFECT OF REACTION SYSTEM

3.1 Introduction

The majority of early works based on experience and some conservative assumptions for 2D numerical model thus gave various recommendations of influence spacing of the reaction system. With the development of high performance PC and powerful finite element analysis tools, it is possible that more complicated models representing the real situations be simulated. In this thesis, numerical analyses of 3D finite element models using PLAXIS 3D foundation were performed with the following objectives:

1) To study the influence of the spacing between test pile and reaction system on the settlement of test pile.

2) To study the influence of geometric factors such as pile diameter, D, length/diameter ratio, L/D, and kentledge width B on the settlement of test pile.

3) To study the influence of soil parameters such as $E_{\text{pile}}/E_{\text{soil}}$ on the settlement of test pile.

The basic geometric parameters used for 3D analyses are listed in Table 3.1 and illustrated in Fig.3.1. Two 3D finite element models representing the pile load test with kentledge and reaction pile system respectively are illustrated in Fig.3.2a and 3.2b. Considering the commonly used pile length/diameter ratio and characteristics soil stratigraphy that one usually meets in practice, only piles with specified L/D ratio in the non-homogeneous soil layer are simulated in this Chapter. Soil layer is assumed to
be stiff to very stiff clay of sedimentary Jurong Formation, say SPT value N changed from 8~10 to 20~25 along the depth. Soil parameters based on the corelation of sedimentary Jurong Formation (Karr Winn et al., 2001) in Singapore and material parameters of pile are listed in Table3.2a. PLAXIS offers the option of an undrained analysis with direct input of the undrained shear strength \( c_u \) and \( \varphi=\varphi_u=0^\circ \) in M-C model (seen in Section 2.4.2.10). The interface element is applied between piles and soil, the strength reduction factor \( R_{\text{inter}} \) is assumed to be 1. The Coulomb criterion is used to distinguish between elastic behavior, where small displacements can occur within the interface, and plastic interface behavior when permanent slip may occur. The definition of the properties and the details about the pile-soil interface are referred to Section 2.4.2.4~2.4.2.7.

<table>
<thead>
<tr>
<th>Table 3.1</th>
<th>Basic Geometrical properties of 3D Models</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (m)</td>
<td>Width (m)</td>
</tr>
<tr>
<td>50</td>
<td>30</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 3.2a</th>
<th>Material properties used in the analyses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td>Model</td>
</tr>
<tr>
<td>Jurong Form.</td>
<td>M-C</td>
</tr>
<tr>
<td>Pile</td>
<td>Elastic</td>
</tr>
</tbody>
</table>

In the kentledge test model, two cribbage pads locate axisymmetrically on the side of test pile over the ground surface to spread the load are simulated with floor element (see in Section 2.4.2.3) in PLAXIS 3D Foundation. The properties of the floor element are assumed to be concrete and listed in the Table 3.2b, in which d is thickness of the cribbage pads.
Table 3.2b  Material properties used in the analyses

<table>
<thead>
<tr>
<th>Element</th>
<th>E(kPa)</th>
<th>G(kPa)</th>
<th>Poisson Ratio</th>
<th>( \gamma ) (kN/m(^3))</th>
<th>d (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor</td>
<td>3E+07</td>
<td>1.25E+07</td>
<td>0.2</td>
<td>24</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Two tension piles make up of the reaction system for another pile load test. These two tension piles are assumed to have equal size to the test pile and are located diametrically on either side of test pile.

According to the empirical equation listed in Code of Practice for Foundation (Singapore Standard CP4:2003), the ultimate axial bearing capacity of the pile can be estimated by:

\[
Q_u = \alpha C_u A_s + N_c (C_u)_b A_b
\]  

(3.1)

where:

- \( Q_u \) is the ultimate bearing capacity
- \( A_s \) is the surface of the pile shaft.
- \( A_b \) is the plane area of the base.
- \( \alpha \) is the adhesion factor, 0.85 is taken.
- \( C_u \) is the average undrained shear strength of the clay.
- \( (C_u)_b \) is the undrained shear strength of the clay at the level of the pile base.
- \( N_c \) is the bearing capacity factor, usually taken as 9.

Because three kinds of pile length are involved, the one in-between, say 20-meter-long pile load test is regarded as the standard pile load test. Considering 2~2.5 times working load, a 5000 kN compression load is determined as the testing load. The reaction forces on reaction systems are different in the two kinds of load test. For the kentledge test, the maximum load on the pile should be such that there is no possibility
of lifting the kentledge off the cribbage. Therefore, a 10 to 20% margin (Weltman, 1980; Tomlinson, 1977) on the calculated weight of the kentledge is incorporated. In the following analysis, the dead weight of 120% of working load is applied, which means 3000 kN of dead load was applied on each cribbage pad at the beginning of the load test. When the pile is loaded in step, the half value of the step load is deducted from the dead load on the cribbage of each side in the same step. On the other hand, in the tension pile system, the compression load on the test pile is increased by step to 5000 kN while each reaction pile shares half of that load in tension to the maximum of 2500 kN.

To measure the influence of the reaction system on the settlement of the test pile under ideal circumstances, the same measurement parameter as that of Poulos and Davis (1980) is adopted. A correction factor, $F_c$, is defined as:

$$F_c = \frac{\rho}{\rho_m}$$

(2.4)

Here $\rho$ is representing the settlement calculated under the ideal test condition without the influence of the reaction system, while $\rho_m$ is measured settlement considering the influence of the reaction system. The correlation of correction factor $F_c$ versus relevant parameters is plotted and studied in the following sections.
3.2 Pile Load Test with Kentledge

3.2.1 General

Several cases are designed to study the influence of the kentledge reaction system on the settlement measurement of test pile. The basic model is that a 1.0-m-diameter test pile under 5000 kN compression load in a finite non-homogeneous Jurong Formation clay stratum, as in Fig. 3.2a. The ratio of thickness of soil layer H to the length of test pile L remains constant of 3. To keep the $E_u/C_u=300$, which can be regarded as a typical ratio of Jurong Formation, the undrained shear strength $C_u$ for the soil layer is increased from 50 kN/m$^2$ with depth by 3.3 kN/m$^2$ per meter, while the undrained Young’s Modulus $E_u$ of soil is increased from 1.5E+04 kN/m$^2$ by 1000 kN/m$^2$/m with depth. The cribbage of the kentledge is spaced on opposite side of the test pile with 2m in width and 6m in length. Because the weight of the kentledge is assigned 20% more than the capacity of the test pile, the weight distributed to each cribbage pad before the pile was loaded should be 3000 kN and then the average pressure on the area of one cribbage pad should be

$$ q = \frac{W}{A} = \frac{3000}{2 \times 6} = 250kPa $$

In which W is the half of the total weight of the kentledge; A is the area of cribbage pad. When the test pile is loaded subsequently, the pressure should be relieved from the pad and half of that increment will be deducted from the pressure of each pad. For example, when load of 5000 kN exerts on the test pile, the pressure of cribbage pad becomes

$$ q_i = q - \frac{W_i}{A} = \frac{3000 - 2500}{2 \times 6} = 41.667kPa $$

(3.1)

Only this two loading procedures are considered in the following calculation.
The S/D ratio, in which S is the distance from center of cribbage to center of test pile, D is the diameter of the test pile, as in Fig.3.1, is varied case by case in the following.

3.2.2 Influence of L/D

To study the influence of L/D, in which L is the length of the test pile and D is the diameter of the test pile, on the settlement of test pile, the diameter of pile is equal to 1m, and L/D ratios of 10, 20 and 50 are investigated. Accordingly, the lengths of piles are 10m, 20m and 50m in the respective models.

The correction factor, $F_c$, is plotted against L/D ratio in Fig.3.3. It is noted that the higher the L/D ratio, the larger is the correction $F_c$, which means the greater is the pile-soil interaction. The FEM results also indicate that the influence of the kentledge is much higher than commonly expected. For example of the L/D ratio of 10, the discrepancy of the settlement would be 18.6% of settlement without the influence of the kentledge even for the S/D=6.

3.2.3 Influence of B

The study on Influence of B, the width of the cribbage pad, is carried out in the model where the test pile is 1m in diameter and 20m in length. The soil condition and the load on the test pile are kept unchanged while the width of the cribbage pad B is changed keeping the cribbage area constant with 12 m$^2$. Therefore, corresponding to the different B of 1.5m, 2m and 3m, the lengths of counterparts are 8m, 6m and 4m.
In Fig. 3.4, it can be seen that narrower cribbage will exert less influence on the settlement of the test pile at a fixed spacing between center lines of kentledge and pile under the same load level given the area of the cribbage remains constant. The influence of the B will become insignificant with the increase of s/d ratio and can be ignored at a distance beyond 5 times the diameter away from the test pile.

### 3.2.4 Influence of Area of Cribbage

Different area of cribbage is studied in the model where the test pile is 1m in diameter and 20 m in length. Under the same load, the bigger the area of cribbage is, the less pressure transferred from the kentledge to the ground around the test pile, which means for the different areas of 16 m$^2$, 12 m$^2$, 8 m$^2$, the initial pressure of the cribbage on the ground are 187.5kPa, 250kPa, 375kPa, and then reduce to 31.25kPa, 41.66kPa, 62.5kPa following the increase of the load applied on the test pile.

The influence of the different area of cribbage on the test pile is illustrated in Fig. 3.5. It is found that the different area of the cribbage has little influence on the movement of the test pile when the spacing is larger than 4D. However, the larger area of the cribbage has the advantage to reduce the influence on the test pile in small spacing less than 4 times the pile diameter.

### 3.2.5 Influence of K

K, pile stiffness factor, is defined as the ratio of Young’s modulus of test pile $E_p$ to Young’s modulus of surrounding soil $E_s$. It is a measure of the relative compressibility
of the pile and the soil. The more relatively compressible the pile, the smaller the value of \( K \).

Two kinds of \( K \) value are simulated in the model where the test pile is 1 m in diameter and 20 m in length. Both loads applied on the test pile are the same to be 5000 kN while the Young’s modulus of test pile \( E_p \) remains constant to be \( 3 \times 10^7 \) kN/m\(^2\); for \( K \) equal to 2000, the average Young’s modulus \( E_s \) of around soil along the pile is equal to \( 1.5 \times 10^4 \) kN/m\(^2\); for \( K \) equal to 4000, the \( E_s \) is 7500 kN/m\(^2\). Meantime, the corresponding \( E_u/C_u \) ratios in both the case remain constant of 300.

The result shown in the Fig. 3.6 indicates that the softer the soil compared to the pile stiffness, the less influence of the kentledge weight on the test pile. The reason is exposed by checking in the plastic point distribution in the finite element results that the softer soil around the pile come into yielding thus produces more plastic settlement compared to the stiffer soil under the same load level. In other words, the greater nonlinearity of the soil structure around the pile reduces the degree of soil-pile interaction.

### 3.3 Pile Load Test with Tension Piles

#### 3.3.1 General

The methodology of study on the influence of tension piles on the result of pile load test is quite similar to that of kentledge system. Two reaction piles with the equal size of test pile substitute for the cribbage pad in the models, as in Fig. 3.2b, the size of the test pile and the soil profile are the same as the kentledge system. However, the load
applied on the reaction system is different in these two kinds of tests. Using the tension piles as the reaction piles, there is no external force like kentledge weight applied on the surrounding soil at the initial test stage. The two tension piles will resist the uplift force that is transferred by the loading beam, when it provides the reaction force for the hydraulic jack between the test pile head and loading beam to push the test pile downward. Hence, given that the reaction piles are located symmetrically on either side of the test pile, the tension force in each reaction pile will be about half of the load on the test pile.

3.3.2 Influence of L/D

To study the influence of L/D, in which L is the length of the test pile and D is the diameter of the test pile, on the settlement of test pile, the diameter of pile is equal to 1m, and L/D ratios of 10, 20 and 50 are investigated. Accordingly, the lengths of piles are 10m, 20m and 50m in the respective models. At this time, the reaction system is two tension piles with equal size of the test pile.

The correction factor, $F_c$, is plotted against L/D ratio in Fig.3.7 Compared with the results of the Poulos and Mattes (1975) seen in Fig.2.11, it is noted that the in the range of spacing between the test and reaction piles commonly used (2.5 to 4 diameters), although the $F_c$ may be larger than 1.4, it is much smaller than that of elastic calculation value of 2 or even greater in Poulos and Mattes (1975). Beside the different numerical analysis methods, the different consideration on the geometry boundary conditions between 3-D and 2-D problem may contribute to this discrepancy. However, the two methods gave a similar trend that for a higher L/D ratio, say 50, the correction factor $F_c$ is larger than lower L/D ratio, say 20, at S/D ratio higher than 4.
but smaller at S/D less than 4. It maybe because that some other factors such as the length of the pile go beyond the influence of the L/D ratio. In general, the higher the L/D ratio, the larger is the correction $F_c$, which means the greater is the pile-soil interaction.

### 3.3.3 Influence of D

The study on the influence of D is carried out by assigning the $D=0.5\text{m}$, 1.0m and 1.5m in the model where the L/D ratio keeps in 20 and H/L ratio keeps in 3. Consequently, the corresponding pile lengths are 10m, 20m and 30m and the depths of the soil layer are 30m, 60m and 90m. The pressures applied on the test pile are the same in all the models and equal to 6366 kPa in order to ensure the soil around the pile bears the equivalent stress level of the standard case, in which load of 5000 kN is applied on the pile with $D=1\text{m}$.

From Fig.3.8, it can be seen that three curves are almost overlapped indicating that different pile diameter seems not to affect the pile-pile interaction when the s/d ratio is fixed. The little discrepancy may be led by different mesh in different model.

### 3.3.4 Influence of Load Level

The study on the influence of load level is carried out in the model that is mentioned above with $D=0.5\text{m}$ and 1.5m while the L/D ratio of 20 and H/L ratio of 3. For the model that D equals to 0.5m, the pressure applied on the test pile are 6366 kPa and 25465 kPa while for the model that pile diameter equals to 1.5m, the pressure applied
on the test pile are 2830 kPa and 6366 kPa. 25465 kPa and 2830 kPa are the values of pressure when 5000kN is applied on the piles of D=0.5m and 1.5m respectively. As 5000kN is designed as the testing load of pile of D=1m in the load test, this same load may be expected to be much higher for the working load of pile of D=0.5m and relatively lower for that of the pile of D=1.5m. Different loads on the pile cause different extents of nonlinearity in soil. Hence, the influence of load level under the elastic-plastic stress state and elastic stress state may be studied in different reaction pile system.

In Fig.3.9, the different load level shows no influence on the movement of the test pile in the case of D=1.5; but higher load level results in stronger influence in small spacing, seen in Fig.3.10. To explain this problem, the pile head movements $\delta$ are normalized by dividing it by pile diameter. Hence, the $\delta/D$ ratios for the case of D=1.5m are 0.5% and 1.1%, 4% and 6.3% for the case of D=0.5m. It may be because in some small $\delta/D$ ratio, the soil behaviour is more like elastic, the soil-pile interaction can be regarded as independent of the stress state of the soil; however, when $\delta/D$ ratio gets higher, high load causes more nonlinearity and soil-pile interaction is less around the test pile.

### 3.3.5 Influence of K

Similar to the case of the kntledge reaction system, two kinds of K value are simulated in the model where the test pile is 1m in diameter and 20 m in length. Both loads applied on the test pile are the same to be 5000kN while the Young’s modulus of test pile $E_p$ remains constant to be $3E+07$ kN/m$^2$; for K equal to 2000, the average
Young’s modulus of around soil $E_s$ along the pile is equal to $1.5E+04$ kN/m$^2$; for $K$ equal to 4000, the $E_s$ is 7500 kN/m$^2$.

The result shown in the Fig.3.11 indicates that the softer the soil compared to the pile stiffness, the less influence of the reaction piles on the test pile. It also can be seen that the extent of the pile-soil-pile interaction influenced by spacing is weakened by increase of $K$, according to the curvature of the plots.

3.4 Conclusions

The parametric study for the influence of the reaction system on the settlement of the test pile was carried out through the true 3D finite element code PLAXIS 3D Foundation in the numerical model. It can be concluded that:

1) The numerical results indicate that the influence of the reaction system on the settlement of the test pile is usually under-estimated in practice. The commonly recommended minimum spacing of 3D–5D between test pile and reaction system may not be enough, as it tends to have greater influence on test pile results than desired.

2) Kentledge system may have larger influence on the test pile compared to the reaction pile system under the same test condition due to the additional weight higher than load capacity of the test pile for safety consideration.

3) For both reaction system, the larger the L/D ratio, the stronger the pile-soil interaction, thus the higher the correction factor; the softer the soil compared to the
pile stiffness (the larger the K), the less the influence of the reaction system on the test results.

4) For the kentledge system, the influence of the width of cribbage is significant for spacing less than 5D; the narrower the width of cribbage, the less the influence on the test results. The influence of the area of cribbage can not be neglected for spacing less than 4D; the larger area of the cribbage has the advantage of reducing the influence on the test pile even at small spacing.

5) For the reaction pile system, the diameter of pile D does not influence the pile-soil-pile interaction for the same spacing ratio of diameter.

6) At lower load levels, the soil behaviour is more like elastic, the soil-pile interaction can be regarded as independent of the stress state of the soil; however, the higher load level causes more nonlinearity and soil-pile interaction is less around the test pile.
Fig.3.1 Geometric Parameters of 3D Model
Fig. 3.2a  3D Model of Kentledge System

Fig. 3.2b  3D Model of Reaction Pile System
Fig. 3.3  Influence of L/D – Kentledge System

Fig. 3.4  Influence of B – Kentledge System
Fig. 3.5  Influence of Area of Cribbage – Kentledge System

Fig. 3.6  Influence of K – Kentledge System
Fig. 3.7  Influence of L/D - Reaction Pile System

Fig. 3.8  Influence of Diameter of Reaction Pile System
Fig. 3.9  Influence of Load Level - Reaction Pile System

Fig. 3.10  Influence of Load Level - Reaction Pile System
Fig. 3.11  Influence of K - Reaction Pile System
CHAPTER 4

FEM STUDY ON O-CELL TEST

4.1 Methodology

4.1.1 Introduction

The idealized finite element models of O-cell pile load test and conventional static load test (for brevity this will hereafter be conventional test) are simulated with PLAXIS 8 and illustrated in Figs. 4.1 to 4.3. Fig.4.1 is the model of the bottom O-cell test where the O-cell is located 0.5m above the pile toe; Fig.4.2 is the model of middle O-cell test where the O-cell is located 5m above the pile toe; Fig.4.3 is the model of conventional top load test. All the tests are simulated in the same Mohr Coulomb soil model with 15 node elements; the meshes are generated axisymmetrically, with the boundary of 50 meters wide and 60 meters deep, which is more than 3 times length of test pile away from the test pile so that the boundary effects can be ignored; the test piles are in the same size of diameter of 1 meter and length of 20 meters. To ensure the precision of the calculation, the meshes of an area of 15 meters wide, 30 meters deep around the test pile are refined. The O-cell part of the piles in the bottom and middle O-cell test are simulated as solid elements about 10cm thick to be in accordance with the extension range of working O-cell. When the O-cell doesn’t work, the material property of this part is still the same as pile; when the O-cell is opened, the material property of this part is de-activated so that the interaction between upward shaft and downward base of pile is de-coupled and minimized. The ground water table is assumed to be the ground level. Table 4.1 lists the geometric properties of the meshes and structures and Table 4.2 lists the parameters of material properties. The interface
element is applied between piles and soil, the strength reduction factor $R_{\text{inter}}$ is assumed to be 1. The definition of the parameters can be referred to Section 2.4.2.

**Table 4.1** Geometrical properties of mesh and structure

<table>
<thead>
<tr>
<th>Width of mesh (m)</th>
<th>Depth of mesh (m)</th>
<th>Diameter of Pile</th>
<th>Length of Pile (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>60</td>
<td>1.0</td>
<td>20</td>
</tr>
</tbody>
</table>

**Table 4.2** Material properties of the FEM model

(To be continued)

<table>
<thead>
<tr>
<th>Name</th>
<th>Type</th>
<th>Model</th>
<th>$\gamma_{\text{sat}}$</th>
<th>$\mu$</th>
<th>$E_{\text{ref}}$</th>
<th>$c_{\text{ref}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jurong Formation</td>
<td>Undrained</td>
<td>M-C</td>
<td>kN/m$^3$</td>
<td>-</td>
<td>kN/m²</td>
<td>kN/m²</td>
</tr>
<tr>
<td>Concrete</td>
<td>Elastic</td>
<td>L-E</td>
<td>20</td>
<td>0.35</td>
<td>10000</td>
<td>40</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$\phi$</th>
<th>$\psi$</th>
<th>$E_{\text{incr}}$</th>
<th>$c_{\text{incr}}$</th>
<th>$\gamma_{\text{ref}}$</th>
<th>$R_{\text{inter}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>0</td>
<td>1000</td>
<td>0</td>
<td>60</td>
<td>1</td>
</tr>
</tbody>
</table>

The test procedures of these three kinds of methods are simulated by the FEM model. The respective results of the tests are compared in the following sections and to check on the validity of the three assumptions that are made to reconstruct conventional head-down curve according to the test data of O-cell load test:

1. The shaft resistance-movement curve for upward movement of the pile is the same as the downward side-movement component of a conventional head-down test.

2. The end bearing load-movement curve obtained from an O-cell test is the same as the end bearing-load movement component curve of a conventional head-down test.

3. The pile is considered rigid and flexible respectively.
4.1.2 Construction of the Equivalent Head-down Load-Settlement Curve

A curve equivalent to applying the load from the top of the test pile can be constructed from the upward movement-shaft resistance curve and the downward movement-end bearing curve. This can be done by determining the shaft resistance at an arbitrary movement point on the shaft resistance curve. When the pile is assumed rigid, the top and bottom move the same amount and have the same movement but different loads. By adding the shaft resistance to the end bearing at the same movement, a single point on the head-down equivalent load-settlement curve is obtained. By repeating this process for different movement points, the head-down equivalent curve is obtained.

However, the shaft resistance component or end bearing component has to be extrapolated if either upper part shaft resistance or lower part end bearing have reached the ultimate state before the other, or sometimes the capacities of the O-cell are exceeded during the loading. One procedure, which is extremely conservative, is to assume that the other component has also reached ultimate and no further load increase occurs as movement increases. Furthermore, hyperbolic extrapolation is usually applied by constructing the linear regression equation to fit the movement/load vs. movement curve of either component which needs to be extrapolated.

By virtue of the application of finite element analysis tools, the load applied by O-cell on either component of the pile can be simulated to be increased continually even if one or the other component has reached its ultimate. For example, when the end bearing of the test pile has been fully mobilized, the finite element model of O-cell can stop increasing the load on the lower part of the pile and continually increases the load
on the upper part of the pile until the ultimate shaft resistance of this part has also been reached.

4.1.3 Elastic Compression

Although most of the time the pile under load is assumed rigid, the influence of the elastic compression on the test results cannot be ignored when the pile is slender or the load is very large. With strain gauges installed along the length of the test pile, the test pile is divided into many segments, which have strain gauges at its top and bottom. This is to facilitate calculating the elastic compression of test pile accurately as the strain gauges provide the loadings at the top and bottom of each segment.

In the O-cell test, with the arbitrarily selected movements, one can deduce the downward load and the upward load from the upward movement-shaft resistance curve and the downward movement-end bearing curve. The addition of these two loads will give the equivalent top load. However, these arbitrarily selected movements are basically upward and downward movements at the location of the O-cell and do not include the elastic compression of the pile under the equivalent top load. The equivalent movement of pile head should be the movement at the location of the O-cell plus the elastic compression of the part between the pile head and the location of the O-cell under the equivalent top loading, providing that the elastic compression between the O-cell and pile toe under the same circumstances is insignificant. The elastic compression of the part between the pile head and the O-cell can not be calculated directly by strain gauges reading because one does not know the actual strain gauges reading corresponding to the equivalent top load. Hence, triangular shaft resistance distribution method is employed to estimate the elastic deformation.
The equivalent top load is calculated using the same method described above. However, this method assumes that the pile experiences a constant shaft resistance such that the equivalent top load at the pile head will decrease linearly till the load value that is exerted on the lower part of the pile. Therefore,

\[
\text{Deformation} = 0.5 \times \text{Length of the pile} \times (\text{Equivalent top load} + \text{Load on the lower part of the pile}) \div (\text{Cross Sectional Area} \times \text{Young’s Modulus})
\] (4.1)

Although it is an improvement to adjust the test data with the estimated value of elastic compression, the equivalent top load-displacement curve derived from this method gives a lower elastic deformation as can be seen from the Fig.4.4. This method is therefore unconservative.

4.2 Shaft Resistance Comparison

To determine the validity of assuming that upward ultimate shaft resistance is equal to the downward ultimate shaft resistance at the same movement, three tests are designed and simulated with a finite element model with PLAXIS 8.0, see Figs.4.1 to 4.3. Assuming that the test pile is rigid, say Young’s modulus of pile is assigned as $10^3$ higher than the real value, every point on the test pile will have the same movement when the load is applied.

One of the advantages of the three “numerical tests” is that they can be designed so that the downward and upward movement of bottom O-cell test and middle O-cell test
and downward movement of conventional top loaded test are about the same. Consequently, the pressure to push the piles of bottom O-cell upward and downward are determined to be 6400 kN/m$^2$ and 1150 kN/m$^2$ respectively; the counterpart of mid O-cell test is 4750 kN/m$^2$ and 2750 kN/m$^2$ and the head-down pressure of the conventional test is 6400 kN/m$^2$. Consequently, the upward and downward movement of the piles in all these tests is about the same as 38 mm.

When comparing the O-cell test to a head-down test, the net uplift loads must be used, because in an O-cell test, the uplift load corresponding to the self-weight value cancels out the self-weight. In contrast, in conventional head-down test, the self-weight is already in the pile at the start of the test and stays in the pile during the entire test. Although the self-weight is usually a small portion of the maximum load, it is considered for the sake of academic interest in this article. The load transfer curve is obtained and so the unit shaft resistance curve and t-z curve under this load condition.

4.2.1 Load Transfer Curve

The numerical results of respective load transfer curves of three tests are displayed in the same Fig. 4.5. As all the tests are carried out in the same soil profile and the $c'$ and $\phi'$ value which are quite relevant to the shaft resistance remain constant along the depth, the load transfer curves of all the tests are approximate to be a line.

The load transfer curve of conventional test shows that the top pressure of 6400 kN/m$^2$ is decreased evenly to about 1200 kN/m$^2$ along the depth; while the load transfer curves of two O-cell tests are broken up into two parts, one is above the O-cell, which
pressure is increased with the elevation getting lower and the other is below the O-cell, which is similar to the behavior of conventional head-down test.

For the bottom O-cell test, the pressure of the O-cell that pushes the pile upward is decreased from 6400 kN/m² along the pile to about zero at the head. Although the load transfer behavior is right reversed, the similarity between the two test results that mentioned above is evidenced by the mirror image of the load distribution curves of the two tests. On the other hand, the pressure that pushes the lower part of the pile downward give the same results of load transfer behavior as that of the conventional test at the same base movement. The downward O-cell pressure is 1150 kN/m², which is almost the same as the value of the conventional test at the same location that 0.5 meter above the pile base.

For the mid O-cell load test, the upper part load transfer curve is coincide with that of bottom O-cell load test, while the lower part load transfer curve is overlapped with that of top loaded test and bottom O-cell test in the shared parts. The upward pressure of O-cell is 4750 kN/m², equal to the pressure that transferred to the same elevation of 5 meters above the pile toe in the bottom O-cell test. The end bearing of mid O-cell test is about 1200 kN/m², approximate to those of conventional test and bottom O-cell test.

4.2.2 Unit Shaft Resistance

The unit shaft resistance is derived from the load transfer curve by dividing the difference of axial load in the unit length by unit perimetric area, so that the mobilized shaft resistance under the different tests can be compared.
Fig. 4.6 illustrates the unit shaft resistance of the three tests. The increase of the shaft resistance along the depth due to the increase of the effective vertical stress $\sigma_v'$ is insignificant as the main factors of $c'$, $\varphi'$ that influence the shaft resistance are constant. However, there is a little difference of unit shaft resistance between the conventional test and two O-cell tests at the upper part of the piles where the movement is equal in value and opposite in direction. The unit shaft resistance in the conventional test is about 0.8~0.9 times of the value in the two O-cell tests, while the curves of two O-cell tests are match indicates that the equal unit shaft resistances are mobilized in both bottom O-cell and mid O-cell tests.

The difference of unit shaft resistance between the conventional test and two O-cell tests tends to be negligible at the lower part where the piles are move downward. In Fig. 4.6, the unit shaft resistance curve of mid O-cell test at the downward part is overlapped with that of the conventional test and so does the curve of the bottom O-cell test.

The difference of unit shaft resistance between the conventional test and two O-cell tests at the upper part will be discussed in Section 4.5.

### 4.2.3 t-z Curve

The t-z curve from the conventional test, bottom O-cell test and middle O-cell test are compared. Fig. 4.7 consists of t-z curve for the point 10 meters below the pile head. Fig. 4.8 consists of t-z curve for the point 1 meter above the pile toe. The difference at the mesh generation of the models and discontinuity of the stress points that were chosen to calculate the shaft resistance may cause the curve not to be so accurate. To
refine the mesh may be help to modify the problem. Nevertheless, the curves of all the tests show the comparable trend that the results from t-z curve would be similar regardless of tests mode.

At the point 10 meters below the pile head, the movement of the pile is upward for the two O-cell tests and downward for the conventional test. As seen from Fig.4.7, two O-cell tests give fairly close result; the shaft shear resistance is developed at the start and reached its ultimate capacity of about 80kN/m$^2$ when the movement of test pile increase to about 15mm. On the other hand, the shaft shear resistance of the conventional test was fully mobilized when the movement of the pile reached 20mm and the ultimate shaft resistance is only 60 kN/m$^2$, about 0.8 time that of O-cell tests. This result agrees well with that from Section 4.2.2.

Fig.4.8 shows the result for the point 1 meter above the toe, where the movement is upward for the bottom O-cell test and downward for the middle O-cell test and conventional test. It can be seen that the curves of middle O-cell test and conventional test are more comparable as both of the tests reflect the same soil - pile interaction at this point. Although the result of bottom O-cell test is influenced by the boundary condition because the point is very close to the loading place, the curve is fairly similar to the other two and the ultimate shaft resistance agrees well with those of middle O-cell test and conventional test.

The difference of the mesh and discontinuity of the stress point that was used to calculate the shaft resistance led to the difference of unit shaft resistance in the results of the three tests. This would be mentioned in Section 4.5.
4.3 End Bearing Comparison

A point at the toe of the test pile was selected and the movement of this point versus the toe bearing was plotted. As the toe bearing was obtained from the stress of this point multiply the toe area of the pile, the boundary effect made the stress on the boundary unstable. To refine the mesh locally at the toe or insert the interface element around the toe may be help to modify this problem. However, the end bearing results of the bottom O-cell test, middle O-cell test and conventional head-down test are compared in Fig.4.9 to confirm the assumption that the end bearing load-movement curve would be about the same for these three tests.

It can be seen that the end bearing – movement curves of the three tests are quite comparable. At the smaller deflection, the end bearing of two O-cell tests increases faster than the conventional test. One possible reason is that for the head-down test, the load transfer from the top to the toe is unable to effectively activate the end bearing and relies heavily on the side shear resistance; the O-cell however is able to immediately activate the end bearing and therefore gives a faster response; the bottom O-cell test can activate the end bearing more easily than middle O-cell test due to the loading position being closer to the toe. As the test progresses, the conventional test is able to activate the end bearing when the shaft resistance becomes fully mobilized, Therefore the end bearing becomes closer to that of O-cell test thereafter.
4.4 Equivalent Head-down Load-Movement Curve

The equivalent head-down load-settlement curve of bottom O-cell test and middle O-cell test are re-constructed refer to the method described in Section 4.1.2, and compared with the load-movement curve of conventional pile load test.

Fig.4.10 shows the load-movement curves of the three tests that assume the test pile is rigid. It can be seen that two O-cell tests give almost the same curve; the soil shows the elastic response before the movement reaches 16 mm and becomes yielded after that, the ultimate capacity of about 7000 kN/mm$^2$ is achieved. The result of conventional test indicates less stiff elastic response before the movement of 16 mm and lower ultimate capacity of about 6400 kN/ mm$^2$ in the end. One possible reason is that the end-bearing is always mobilized more lately at the same displacement for the conventional test than the O-cell tests.

Fig.4.11 gives the load-movement curves of the three tests that consider the elastic compression of the test pile using the triangular method recommended in Section 4.1.3. It can be seen that the curve of bottom O-cell test gets closer to that of conventional test at most part but shows a little higher capacity at larger movement, while the adjustment due to the elastic compression is not so obvious for the curve of the mid O-cell test. One reason may be that for the mid O-cell test, the elastic deformation below the O-cell should not be neglected as the lower part of the pile is much longer than that of bottom O-cell test. However, the conclusion that two O-cell tests will give the larger ultimate capacity than conventional test can be achieved for both cases whether considering the elastic compression or not.
4.5 Drained Analysis

To explore the reason why simulation of PLAXIS 8 gives higher ultimate capacity for the O-cell tests than the conventional test, three models are made using drained analysis in which the soil properties are assigned to be drained. For the majority of pile loading tests, the time of loading procedure is always transient compared to the consolidation period of the clay soil. Therefore, undrained analysis is correctly applied to simulate the short-term behavior of pile loading test but drained analysis would correctly give the long-term soil-pile interaction in the same soil profile, when all soil consolidation effects have been completely dissipated.

The load-transfer curves, unit shaft resistance curves and equivalent head-down load-settlement curves of three tests under drained condition are displayed in Fig.4.12 to 4.14.

In Fig.4.12, the more symmetrical load-transfer curves of O-cell tests and conventional test at about the same amount of movement further evidenced that O-cell tests can be seen equivalent to the conventional test. The overlapped curves of bottom O-cell test and middle O-cell test indicate that the test results are not influenced by the location of the O-cell, so that the O-cell can be installed in any proper depth where the upper part and lower part of the test pile can provide the matching reaction force for each other, whereby the full capacity of O-cell can be put to good use.

The equivalent head-down load-settlement curves of three tests showed in Fig.4.14 indicate that the O-cell test can give the same ultimate capacity of the pile as the conventional one providing that long-term effects is considered. It is further suggested
that the O-cell test can be a good substitute for the conventional test to investigate the soil-pile interaction in practice.

Besides, the results of drained analysis raised the issue of the influence of excess pore pressures in interpreting the difference of ultimate capacity of the three tests under undrained analysis. In undrained analysis, full development of excess pore pressures is involved. In different modes of loading for the three tests, with different loading direction and position and different boundary conditions, different sets of excess pore pressures around the pile are generated when the pile is compressed from the top or uplifted from the base. Accordingly the radial effective stresses along the pile in the three tests are different, which contribute the major part of the differences in the shaft resistance obtained in the different pile test systems. On the other hand, drained analysis is applied to simulate long-term soil behavior without involving excess pore pressures and consolidation of the soil, thus the results reflect that the shear resistance is independent of direction of shear for all practical purpose (Fellenius, 2002).

Fig.4.13 indicates that the mobilized unit shaft resistance acting in the negative direction for the two O-cell tests is quite closer to that acting in the positive direction for the conventional test under the same deflection compared with the corresponding results of the drained analysis.

4.6 Conclusions

In this chapter, three kinds of pile load test including conventional head-down test, bottom O-cell test and middle O-cell test have been studied with PLAXIS 8 in identical geotechnical conditions. Several conclusions can be drawn and further study
indicates that the possible reasons that may lead to some discrepancy among the three tests results.

1) It is concluded from the preceding FEM study that O-cell test result can provide not only the same soil-pile interaction information as conventional head-down static loading test, but also allow for separate determination of the shaft resistance and end bearing components.

2) The FEM computation indicates that the shaft resistance-movement curve for upward movement of the pile is fairly comparable with the downward shaft resistance-movement component of a conventional head-down test. The result of undrained analysis suggests that a little difference of mobilized shaft resistance may exist between O-cell test and conventional test in the short-term view due to the different sets of excess pore pressures generated. However, this discrepancy will attenuate in the long-term.

3) Also the end bearing load-movement curve obtained from an O-cell test is quite close to the end bearing-load movement component curve of a conventional head-down test. The curve of O-cell test may reflect a stiffer load-movement response than that of conventional test probably due to the shorter load transfer distance between pile toe and point of loading application, so that the end bearing of the O-cell test is mobilized much earlier compared to conventional test.

4) The equivalent head down load-movement curve of the O-cell test simulated by PLAXIS 8 gives a little stiffer load-movement response and slightly higher
ultimate capacity than those of conventional test. Apart from the computational schemes of the FEM program itself, the differences of effective stress around the pile due to the different excess pore pressures generated from the different load-transfer mechanism among these kinds of pile load tests contributed to the discrepancy of unit shaft resistance of these test piles under the same pile movement. When the drained analyses were made and long-term soil-pile interaction was considered, both the O-cell test and conventional test gave very close results.

5) The elastic compression of the pile could be considered for an O-cell test so that the more agreeable results with that of conventional test can be obtained. However, the triangle method recommended in this thesis will give a slightly unconservative estimate.

6) The O-cell level has essentially no influence on the test results for a uniform soil. In this thesis, the FEM analysis indicates that bottom O-cell test and middle O-cell test would give the same results. In practice, the O-cell level should be determined based on adequate resistance from the lower portion of the pile (end bearing plus shaft resistance of lower portion) to counteract the upper resistance of the pile.
Fig. 4.1 FEM Model of Bottom O-cell Test
(2063 15-node triangular elements)
Fig. 4.2  FEM Model of Middle O-cell Test
(2063 15-node triangular elements)
Fig. 4.3  FEM Model of Conventional Static Pile load Test
(2063 15-node triangular elements)
Fig. 4.4  **Calculation of Elastic Compression using Triangular Side Shear Distribution**

Fig. 4.5  **Comparison of Load-Transfer Curves**
Fig. 4.6 Comparison of Unit Shaft Resistance Curves

Fig. 4.7 Comparison of t-z Curves at EL.10m
Fig. 4.8 Comparison of t-z Curves at EL. 19m

Fig. 4.9 Comparison of End-Bearing Curves
Fig. 4.10  Comparison of Load-Movement Curves (Rigid Pile)

Fig. 4.11  Comparison of Load-Movement Curves (Flexible Pile)
Fig. 4.12 Comparison of Load-Transfer Curves (Drained)

Fig. 4.13 Comparison of Unit Shaft Resistance Curves (Drained)
Fig. 4.14  Comparison of Load-Transfer Curves (Drained)
CHAPTER 5

CASE HISTORY 1: PILE PTP1 IN GOPENG STREET PROJECT

5.1 Introduction

5.1.1 General

A proposed development comprises two residential tower blocks, one is 46-storey, the other is 48-storey, was constructed on approximately 6566.6 m² site at Gopeng Street, Singapore, seen in Fig.5.1. The proposed site and the surrounding regions are underlain by Alluvial Member (Ka) of Kallang Formation and Queenstown Facies (Jq) and Rimau Facies (Jr) of Jurong Formation.

Shallow foundation scheme (e.g. raft or footings) is considered to be inadequate in supporting the structural loads of the proposed towers due to the relatively low shear strength and relatively high compressibility possessed by these upper soil layers. Therefore, the use of deep foundation (i.e. pile foundation) to transfer the structural loads through the incompetent soil layers to deep hard layers is considered necessary.

Due to the inherent non-conformity of ground conditions encountered, two O-cell test on the preliminary test piles named PTP1 and PTP2 to verify design assumptions (e.g. end bearing and shaft resistance of pile embedded in very dense or hard soil), pile performance and adaptability of installation method proposed by piling contractor, are recommended to be carried out prior to the installation of working piles. Only the O-cell test of PTP1 is described and back-analyzed in detail in this chapter.
5.1.2 Study Objective

This case history was back-analyzed with the following objectives:

1) To model the O-cell test with PLAXIS 8 and produce the matchable test result.
2) To utilize the same FEM model verified by back-analysis of existing O-cell test to extrapolate the load-movement component of either upper portion or lower portion of the O-cell test pile.
3) To compare the results of the O-cell test with those of conventional test when simulated in the same FEM model.

5.2 Field O-cell Test

5.2.1 Instrumentation Description and Geotechnical Condition

Fig. 5.2 presents the schematic section of PTP1. The elevation of ground surface at the test site is EL.104.27 m. The 1200mm-diameter test pile PTP1 was constructed wet using bentonite slurry support with top elevation of EL.104.84 m and tip elevation of EL.80.49 m; the embedded length of the test pile in the soil was 23.78m. The temporary 1230mm-diameter O.D. casing between EL.104.84 and EL.102.84 was removed after concrete placement. The O-cell assembly (four 400-mm O-cells) was located 2.35 m above the tip of pile. The O-cell assembly has a maximum test load capacity of 16.6MN and a maximum plate displacement of 32.9mm.

Standard O-cell testing instrumentation included two numbers of telltale rod extensometers fixed to the bottom steel bearing plate to directly measure O-cell expansion minus compression. Bottom plate telltale movements and pile toe telltale movements were monitored by two LVWDTs (Linear Vibrating Wire Displacement
Transducers) each at the top of the pile. Compression of the pile above the O-cell assembly was measured by three telltales. Telltale movements were monitored by LVWDTs at the top of the pile. Two LVWDTs attached to a reference beam monitored the top of pile movement. One level of four sister bar vibrating wire strain gages were installed in the pile below the O-cell. Seven levels of sister bar pairs of vibrating wire strain gages (VWSGs) were installed in the pile above the O-cell assembly. Details concerning the strain gage placement are in Fig.5.2.

According to a borehole log near the test pile (Appendix A), the sub-surface stratigraphy at the test pile location consists of clayey silt up to a depth of 2 m underlain by 4 m thick of clayey sand. Between depths 6 m and 10 m, the soil is identified as silty sand with very weak siltstone fragments. From depth of 12 m to 17.5 m, silty sand with siltstone fragment was encountered. Very dense silty sand with siltstone fragment was identified between depths of 17.5 m and 20 m. The material below depth of 20 m to the pile toe consists of slightly weathered siltstone and completely decomposed siltstone. The groundwater table was about 2m below the ground surface on the average.

5.2.2 Test Procedure

The O-cell load test was conducted as follows: The four 400-mm diameter O-cells, with their base located 2.35 m above the tip of pile, were pressurized to assess the combined end bearing and lower shaft resistance characteristics of the pile section below the O-cells and the upper shaft resistance above the O-cells. The O-cells were pressurized using the Quick Load Test Method for Individual Piles (ASTM D1143 Standard Test Method for Piles under Static Axial Load) in 34 equal pressure
increments to 46.88 MPa resulting in a bi-directional gross O-cell load of 16.6 MN.
Each successive load increment was held constant for 15 minutes by an automatic pressure maintenance unit which maintains the desired O-cell pressure.

5.3 Back Analysis

5.3.1 General Settings

The numerical simulation of the O-cell load test was performed through the finite element program PLAXIS 8. In the analysis for both the O-cell test and conventional head-down test, the following settings were assigned and some assumptions were made:

1) Axisymmetrical model was adopted considering the boundary conditions of the pile load test.

2) Mohr Coulomb failure criterion was used for most of the soil types. Hardening soil model was applied to simulate the behaviour of slightly weathered siltstone and completely decomposed siltstone below the pile toe.

3) Interface elements were incorporated along the pile to simulate the soil-pile interaction and extend 0.5 m beyond pile toe to reduce the corner effects causing undesirable high peaks in stress and strains around pile toe.

4) The O-cell is simulated with a 10-cm-thick solid element. When the O-cell was close, the solid element was assigned the properties of concrete; when the O-cell was working, the property was changed to a fictitious elastic material with very low stiffness so that the upward and downward movement of the pile would not be influenced by its displacements.
5) According to the geotechnical description of the borehole log, all soils are clayey/silty materials and behaviour of all the soil strata can be assumed to be undrained.

6) Most of the soils can be regarded as normally consolidated according to the laboratory consolidation test result, although some overconsolidation of the stiffer soils may be possible.

7) No dilatancy effect of the soil was considered.

8) Construction rate was assumed to be faster compared to consolidation rate of soils.

9) The elastic compression of the pile is taken into account.

Therefore, a 20-meter-width, 40-meter-high axisymmetrical finite element model consisting of 374 fifteen-node triangular elements and 3181 nodes was used and illustrated in Fig.5.3. In the local area of 6-meter-width, 28-meter-high around the test pile, the mesh was refined to ensure the precision of the calculation. The average element size was 1.46m².

5.3.2 Material Properties and Soil Profile

The ACI formula \( E_c=57000 \sqrt{f_{c'}} \) was used to calculate an elastic modulus for the pile concrete, in which \( f_{c'} \) was the concrete unconfined compressive strength and was reported to be 44.0 MPa. This, combined with the area of reinforcing steel and nominal pile diameter, provided average pile stiffness (EA) of 44200 MN in the upper cased portion of the pile, 33300 MN above the O-cell and 32700 MN below the O-cell according to the data reports provided by the contractor.
Refer to the installation level of strain gage (Fig.5.2) and nearby borehole log (Appendix A), the eight layers of soil profile were simulated in the PLAXIS model, as seen in Fig.5.3. As no field test data on the effective soil properties such as $c'$ and $\phi'$, undrained analysis with direct input of the undrained shear strength ($C_u$) and $\phi=\phi_u=0$ are available for the hardening soil model (referred to Section 2.4.2.10). Undrained soil parameters were determined here according to the field investigation and widely accepted correlations in local practice. The values of ultimate unit shaft resistance $q_s$ that were measured from the instrumented test are presented in Table 5-1, which were used to deduce the soil parameters that needed for the finite element back-analysis.

According to the Semi-empirical methods that have been proposed by Burland (1973) and Meyerhof (1976), the ultimate unit shaft resistance, $q_s$, on the pile shaft can be calculated by equation:

$$q_s = \alpha C_u$$

(5.1)

where $\alpha$ is an adhesion factor, which can be referred to in Fig.5.4 (Weltman, Healy 1978) for the cast-in-place bore pile, $C_u$ is the average undisturbed undrained shear strength of the soil surrounding the pile shaft and here can be estimated by inverted equation 5.1. The correlation between $E_u$ and $C_u$ for the normally consolidated clay is summarized by Duncan and Buchignani(1976) in Fig.5.5. For the slightly weathered siltstone and completely decomposed siltstone below the pile toe, a hardening soil model was applied. Equations that are introduced in Section 2.4.2.7 were used to estimate the other soil parameters of the hardening soil.
The soil properties that were adjusted according to the comparison on back-analysis result and measured data to get the best fit are summarized in Table 5.2 together with the pile concrete characteristics.

5.3.3 Construction Stages

There were 34 levels of loading increment performed in the O-cell test. For the study purpose, only four levels were simulated in the model. The FEM model construction sequence for the O-cell load test was as follow:

1) Initial state of soil
2) Pile installation
3) L-08: O-cell is loading to 3.95MN
4) L-16: O-cell is loading to 7.85MN
5) L-24: O-cell is loading to 11.76MN
6) L-34: O-cell is loading to 16.63MN
7) Unloading to zero

<table>
<thead>
<tr>
<th>Table 5.1</th>
<th>Average Net Unit Shaft Resistance for 1L-34</th>
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<tbody>
<tr>
<td>Depth (m)</td>
<td>Load Transfer Zone</td>
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<tr>
<td>0.00 – 4.53</td>
<td>Zero Shear to Strain Gage Level 8</td>
</tr>
<tr>
<td>4.53 – 6.53</td>
<td>Strain Gage Level 8 to level 7</td>
</tr>
<tr>
<td>6.53 – 9.53</td>
<td>Strain Gage Level 7 to level 6</td>
</tr>
<tr>
<td>9.53 – 12.53</td>
<td>Strain Gage Level 6 to level 5</td>
</tr>
<tr>
<td>12.53 – 15.03</td>
<td>Strain Gage Level 5 to level 4</td>
</tr>
<tr>
<td>15.03 – 17.43</td>
<td>Strain Gage Level 4 to level 3</td>
</tr>
<tr>
<td>17.43 – 21.43</td>
<td>Strain Gage Level 3 to O-cell</td>
</tr>
<tr>
<td>21.43 – 23.78</td>
<td>O-cell to Strain Gage Level 1</td>
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</table>

* For upward-loaded shaft resistance, the buoyant weight of pile in each zone has been subtracted from the load shed in the respective zone
### Table 5.2  Material Properties of PTP1 in PLAXIS 8

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Symbol</th>
<th>1-Medium clayey silt</th>
<th>2-Medium dense clayey silt</th>
<th>3-Very dense silt/ sand</th>
<th>4-Very weak siltstone fragments</th>
<th>5-Very dense silt/ sand with siltstone fragments</th>
<th>6-Very dense silt/ sand with siltstone</th>
<th>7-Moderately weathered siltstone</th>
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<td>Material model</td>
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<td>M-C</td>
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<td>Type of behaviour</td>
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<td>110</td>
<td>600</td>
<td>260</td>
<td>450</td>
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<td>Friction angle</td>
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<td>0</td>
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<td>R_inter</td>
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</tr>
</tbody>
</table>

*--do not take into consideration
5.4 Results and Discussion

5.4.1 Load-Movement Curves

The back-analyzed load-movement curves are compared with measured one in Fig.5.6. It is clear that the finite element model can give a satisfactory simulation to the displacement of the pile under the loading of O-cell.

The maximum upward and downward applied load, maintained for 15-minutes, is 16.6 MN which occurred at load interval 1L-34. At this loading, the average downward movement of the O-cell base is 32.9mm; the average upward movement of the O-cell top is 15.3mm. While the calculated maximum upward and downward movements in FEM model are 32.7mm and 14.8mm respectively, which are quite close to the measured ones.

The measured upward and downward residual movements are 11.95mm and 28.08 mm, while the calculated downward residual movement is 27.3mm, which agree well with the measured. However, the result of upper part of the FEM model indicates more elastic response; the upward residual movement is 6 mm, one half of the real value. This is probably due to the reason that Mohr-Coulomb model is applied to simulate the behavior of the soil layer around the pile while Hardening-Soil model is applied to the soil layer below the pile toe. The M-C model is an elastic perfectly-plastic model with fixed yield surface, so that for stress state within the yield surface, the behavior is purely elastic and all strains are reversible. On the other hand, the H-S model considers the decreasing stiffness and simultaneously irreversible plastic strain develop during the loading on the soil.
5.4.2 Load-Transfer Curves

The Load-Transfer curves of four loading levels are listed in Figs.5.7 to 5.10. The Load-Transfer curves show that the upper portion of the pile bears little shaft resistance but more loads is carried by deep soil layer; load exerted from O-cell is decreasing along the pile far away from the place of loading application. At the head of the pile, the up-lift force decreases to zero due to the offset of negative skin friction by soil-pile interaction. While at the other tip of the pile, the push-downward force is decreased by positive skin friction similar to the load transfer mechanism of conventional test to end-bearing value of the pile base.

From Fig. 5.7 to 5.10, it can be clearly seen that FEM analysis agrees well with the actual field measurements. The difference between the calculation and measurement becomes small when the load level increase. This should be because all the soil parameters are deduced according to the soil behavior under ultimate loading state. If more complex soil model such as Hardening soil model is used, which can capture the non-linear loading stress strain variation, more accurate results can be achieved. However, Hardening soil model requires more soil parameters, which are difficult to get when the field test data are scarce. Besides, M-C model can give the satisfied simulation in most of the cases.

5.4.3 Unit Shaft Resistance Curves

The calculated unit shaft resistances of the test pile under the four load levels are compared with the corresponding measured result, shown in Figure 5.11 to 5.14.
respectively. Very close match indicates that the FEM model can perform a satisfactory simulation to the field test if the sufficient information is provided to determine the soil parameters. According to the result, the upper layer soil above EL.92m is very weak compared with that below. At the depth of EL. 92m to EL. 89m, there is a very strong soil layer, which corresponded to the silty sand with siltstone fragment layer with SPT value higher than 100 in the nearby borehole log. However, the soil layer below the EL. 85m can provide the maximum unit shaft resistance.

The similar trend in difference between the measured and calculated result as that in load-transfer curve is seen in the unit shaft resistance curves under the four load levels. However, the accuracy of the calculation would be compromised given that complicated soil layers are simulated and some simplified assumptions are made in the soil behavior.

5.4.4 FEM Extrapolation

As the O-cell load test had to be stopped because the capacities of O-cell had been exceeded, the end bearing and shaft resistance of parts of pile above or below the O-cell may not fully mobilized. Accordingly, the equivalent head-down load-movement curves that are generated by using the measured upward top of O-cell and downward base of O-cell data may not cover all the involved component of the load-movement curve that would be important in practice. Hence, some methods to extrapolate either component of O-cell load-movement curves are required to produce top loading load-movement curve as in conventional pile load test.
As the general method of using hyperbolic curve fit to obtain the equation of the trendline of load-movement curve doesn’t take any geotechnical view into consideration, it is strongly believed that using back-analyzed FEM model to fulfill the extrapolation is more rational and reasonable than hyperbolic curve fitting.

In this case, both the components of O-cell load-movement curves are extrapolated by easily exerting the up-lift force to 18.8 MN and the push-downward force to 18.0 MN in the same FEM O-cell model. The load-movement curves after extrapolation are shown in Fig.5.15. It can be seen that the capacity of the shaft resistance of upper part has been reached when loading to 18.8 MN while the end bearing of lower part is still increasing at load of 18.0 MN.

5.4.5 Equivalent Conventional Test

To compare the equivalency of the O-cell test to conventional head-down static loading test, a conventional test is simulated in the same FEM model except that the pile was loaded pushing downward from the pile head.

Using the method recommended in Sections 4.1.2 and 4.1.3, a conventional head-down load of 28.3 MN that is equivalent to the L-34 O-cell load was simulated in the FEM model.

Fig.5.16 shows the load-transfer curve of the conventional test together with those of calculated and measured O-cell result. Comparing those portions above the O-cell level, the similarity between the conventional test and O-cell test is evidenced by a pair of mirror images of the load-transfer curve of the two tests.
Fig. 5.17 presents the unit shaft resistance distribution for the pile as computed for the two types of tests. The shaft resistance acts in the negative direction for the O-cell test and in the positive direction for the head down test. Generally, there is very little difference between the computed unit shaft resistance values for the two types of tests.

The load-movement curve is illustrated in Fig. 5.18 together with the equivalent head-down load-movement curves of the O-cell test, one is extrapolated by hyperbolic curve fit recommended by the contractor, and the other is using FEM extrapolation with PLAXIS 8.

Three curves are almost overlapped before the pile head is settled 25 mm. The results of the FEM simulation for both O-cell test and conventional test indicate that for one time working load of 8.5 MN, the pile would settle approximately 6 mm and for the two time working load of 17.5 MN, that value would be 15 mm, which are very close to the corresponding adjusted test results of 5.8 mm and 13.9 mm considering the elastic compression of the test pile.

After that, the hyperbolic curve fit method doesn’t show a definite yield trend and gives the most strong soil-pile interaction behavior at the end phase; while the two finite element simulations reflect a yield trend. FEM extrapolation curve of O-cell test displays a little stiffer load-movement behavior than that of conventional test in PLAXIS 8, which is quite similar to the conclusions that are drawn in Chapter 4. As it is believed that the difference due to excess pore pressure changes in the two methods of loading piles causes the discrepancy of load-movement behavior, the excess pore
pressure $u_{\text{excess}}$ and effective normal stress $\sigma_n$’ distribution in the interface elements for the 2 cases are illustrated in Fig.5.19 and Fig.5.20. It can be seen that most part of the distribution of the excess pore pressure $u_{\text{excess}}$ for the O-cell test is positive, which means suction effect is taken on the pile surface. On the other hand, head-down loading method makes the distribution of the excess pore pressure $u_{\text{excess}}$ negative. Therefore, under the condition of approximate constant total normal stresses on the pile surface for the two loading methods, the effective earth pressure applied on the pile surface is higher for the O-cell test than conventional head-down test (Fig. 5.20) according to Terzaghi’s principle (effective normal stress and pore pressure are considered negative for pressure in PLAXIS)

However, it can be concluded that the O-cell test would be fundamentally equivalent to a conventional test given that the error of the numerical method applied in the FEM code and the little difference in FEM mesh generation may have influence on the results.
Fig. 5.1  Location of Case Study in Gopeng Street

Fig. 5.2  Instrumentation of PTP1
Fig. 5.3  FEM Model of PTP1
Fig. 5.4  Adhesion Factors for Bored Pile
(after Weltman and Healy, 1978)

Fig. 5.5  Plate Loading Test by Duncan and Buchignani (1976)
**Fig. 5.6**  Comparison of Load-Movement Curve

**Fig. 5.7**  Comparison of Load-Transfer Curve at 1L-8
Fig. 5.8  Comparison of Load-Transfer Curve at 1L-16

Fig. 5.9  Comparison of Load-Transfer Curve at 1L-24
Fig.5.10  Comparison of Load-Transfer Curve at 1L-34

Fig.5.11  Comparison of Unit Shaft Resistance of Curve at 1L-8
Fig. 5.12  Comparison of Unit Shaft Resistance of Curve at 1L-16

Fig. 5.13  Comparison of Unit Shaft Resistance of Curve at 1L-24
Fig. 5.14  Comparison of Unit Shaft Resistance of Curve at 1L-34

Fig. 5.15  Extrapolation of Load-Movement Curve by FEM
Fig. 5.16  Comparison of Load-Transfer Curve of O-cell at 1L-34 with That of Equivalent Conventional Test

Fig. 5.17  Comparison of Unit Shaft Resistance Curve of O-cell at 1L-34 with That of Equivalent Conventional Test
Fig. 5.18  Equivalent Top Load-Movement Curves
Fig. 5.19  Comparison of Distribution of Excess Pore Pressure

Fig. 5.20  Comparison of Distribution of Effective Normal Stress (negative for pressure in PLAXIS)
CHAPTER 6

CASE HISTORY OF STATIC LOADING TEST

6.1 Case History 2: Harbour of Thessaloniki Project

6.1.1 General

The numerical simulation of a case history of the pile load test was performed by Comodromos et al. (2003) through the finite difference code FLAC 3D to assess the axial pile group response based on the field data of the test. In order to evaluate the influence of the interaction between test pile and reaction piles in the realistic field test and compared with the findings mentioned in Chapter 3, the finite element re-analysis is performed using PLAXIS 3D Foundation. For this purpose, the back analysis of a pile load test was initially performed which facilitated the determination of the single pile response and the verification of the soil properties. Subsequently, a numerical analysis was carried out to establish load-displacement relationships for different layouts of pile groups. The numerical results were compared with those of Comodromos et al. (2003).

The project was carried out in the area of the new wharf at the harbour of Thessaloniki in Greece. Based on the preliminary design using the German code DIN 4014 and a working load of 4500kN, the required length and diameter of the bored piles were 45m and 1.5m respectively. The pile load test arrangement included the test pile and four reaction piles system distributed in the cross-shaped location around the test pile. The piles used in the test and reaction tension piles were identical to the foundation piles in length, diameter and reinforcement except that the reinforcement of the reaction piles was modified to undertake the large tension forces. The compressive load was applied
using six hydraulic jacks having a capacity of 2500 kN each, which were placed between the pile head and a cross shaped reinforced concrete beam, as shown in Fig.6.1. Then the reaction was transferred evenly through the concrete cross beam to the four tension piles which were placed at a centre-to-centre distance of 6.0m from the test pile, equivalent to 4 diameters of pile.

The soil profile was determined based on the results of a detailed geotechnical investigation including a Cone Penetration Test, a 50 m deep borehole and laboratory tests on samples taken using Shelby tubes. Fig.6.1 illustrates the soil stratigraphy consisting of four layers. The upper layer, assigned as layer 1, consists of 6-meter soft clayey silt with thin layers of sand, underlying highly plastic soft clay, layer 2, about 12-meter thick. Under the latter layer, at the level of -18.0 m a layer of medium stiff clay, layer 3, of medium plasticity was located, extending down to -42.0 m. From that level to the end of the borehole, a very dense sandy gravel with a clay layer was detected. The main soil properties of each soil layer derived by the geotechnical investigation are also presented in Fig.6.1. The ground water table was measured to be near the ground level.

The maintained load type (ML) was applied in the test and loading sequence adopted in the test including loading, unloading and reloading to the working load level. In detail, the testing sequence include an initial loading up to 4 MN and then unloading in steps of 1 MN; then a second loading/unloading cycle to 10 MN using the same loading and unloading steps; finally a third loading/unloading cycle to the maximum capacity of the hydraulic jacks (15 MN) was applied in step of 1.07 MN and unloading steps of 2.14 MN. The duration of each loading step was sufficient to reach a stable
settlement. In general, a rate of movement of 0.25mm/hr may be taken as the limiting rate for practical purpose.

6.1.2 Back Analysis

The numerical simulation of the pile load test is performed in this section by the finite element code PLAXIS 3D Foundation.

A finite element mesh including 5 piles in a cross-shaped layout is prepared in Fig.6.2. The volume of the model is $41 \times 41 \times 70 \text{ m}^3$, almost 3 times away from the research objects so that the effect of the boundary may be neglected. In an square area of $15 \times 15 \text{ m}^2$ that includes the test pile and four reaction piles, the mesh are refined to satisfy the precision of the calculation. The five piles are labeled P2, P8, P4, P6, and P5, corresponding to the four reaction piles and the central pile under compression respectively, which means each reaction pile carry one quarter of the load applied on the central test pile. In total 4536 elements and 13283 nodes are included in the pile test model. The average size of 15-node wedge element is 5.48 m$^2$.

The elastic perfectly plastic Mohr-Coulomb constitutive law has been used to simulate the non-linear elasto-plastic material behaviour of the soil layers shown in Fig.6.2. Despite the fact that more sophisticated elasto-plastic constitutive models such as Hardening-Soil model exist, which can combine the shear failure surface with a cap surface to simulate soil dilation or compression, the simpler Mohr-Coulomb model was satisfactory enough considering that the anticipated stress paths are mainly
dominated by shear failure for the static pile load test such as in this case. The soil properties of the four soil layers are taken average in Table 6.1 according to the data provided by Comodromos et al. (2003).

**Table 6.1** Soil and concrete properties

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<th>Materials</th>
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<th>Young’s Modulus E (MPa)</th>
<th>Poisson ratio ν</th>
<th>Frictional angle φ(deg)</th>
<th>Undrained shear strength cu (kPa)</th>
<th>Unit weight γ (kN/m³)</th>
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The simulation sequence included an initial step in which the initial stress condition is established and then the installation of five piles, followed by 3 loading/unloading circle just like the real loading situation except the loading/unloading steps is combined to one due to the capability of the program and considering that the stress independent stress-strain behaviour of the M-C model. The predicted load-movement curve is presented in Fig.6.3, together with the finite difference result of Comodromos et al. (2003) and measured field test result. It can be concluded that FEM simulation can give a reasonable match to the FD result and measured result, although the FEM simulation is applied with a very coarse mesh due to the program’s limits of the current version.

In order to determine the effect of the reaction pile system on the behaviour of the test pile, the same loading sequence is applied to the single central pile while the other four reaction pile is deactivated in the same numerical model as the preceding one. The load-movement relationship is established in a curve to compare with that of the test
result in Fig.6.4. It can be seen that there is a remarkable difference between the load-movement curves of the test pile and the single pile for load levels lower that 15 MN.

For example, when the applied load is 4 MN, the single pile settles 7.87 mm while the test pile under the four-tension-pile reaction system settles only 5.18 mm, the correction factor $F_c$ is about 1.52. It is noted that this value agrees very well with the value obtained from Fig.3.7 according to the L/D ratio of 30. However, when the applied load reaches its load capacity of 15 MN, the settlement of the single pile under this load is 41.24 mm while that of the test pile is 32.59 mm. The correction factor $F_c$ reduces to 1.27. The influence of the reaction piles on the test pile is decreased when the load level applied on the test pile increases.

The influence of the different numbers of tension piles is also studied in the same numerical model shown in Fig.6.5. During this simulation, all the steps are following the four-pile reaction system except only P2 and P8 piles are activated to carry the reaction tension load, which means each tension pile carry one half of the load applied on the test pile. The load movement curve is displayed in Fig.6.6 together with that four reaction pile system and single pile test. It is concluded that the influence of the two reaction piles on the test pile is a bit less than that of the four reaction piles under the same load level. The curves of correction factor vs. load level of both are compared in Fig.6.7.

Despite of the limitation of function in the current version of PLAXIS 3D Foundation, the similar conclusions can be drawn from the results of re-calculation of the above case history that:
1) The level of interaction between test pile and reaction piles appearing in pile load tests with commonly used spacing of 3-4 diameters is very important and affects significantly the load-displacement relationship.

2) Interaction between the test pile and the reaction piles is minimal provided that the applied load is sufficiently large to cause significant settlement.

3) The interaction effect would be greater as the number of the reaction piles increases.

6.2 Case History 3: NTUC Project

6.2.1 Study Objective

Usually in practice, conventional pile load test was back analyzed with FEM without considering the effect of the reaction system. In another word, pile load test was regarded as an ideal static load test where the pile is subjected to “pure” vertical loading. However, the objectives of the study on the following case history were:

1) To model the conventional load test with PLAXIS and produce matchable test result.

2) To take the kentledge weight into consideration when simulating the pile load test and compared the load-transfer curves, load-movement curves etc with those of ideal static load test.
6.2.2 General

The proposed NTUC building was located at Raffles Quay/Marina Boulevard. The preliminary test pile was a bored pile with nominal diameter 1500mm installed to a depth of 22.08m below ground level. The pile was cast with concrete of Grade 40. As the concrete of the pile was just 13 days old at the start of the loading test, the actual strength of concrete was considered to be Grade 35. The test pile was designed for a working load of 13200kN. However, the maximum test load of 39600kN, 3 times working load, was performed during the test, the corresponding settlement of the pile head was 45mm. The borehole for the pile was made by rotary auger, after which the steel cage with the vibrating wire strain gauges/sensors and tell-tale extensometers were lowered into the borehole and the concrete placed by tremie. A temporary steel casing (12m) was used during pile construction to avoid negative skin friction.

6.2.3 Instrumentation Description and Geotechnical Condition

Sixteen vibrating wire strain gauges were installed in the test pile at seven different levels corresponding to the soil profile deduced from the nearest borehole log. These were installed as shown in Fig. 6.8. Two tell-tale extensometers were installed in the test pile. Two linear vertical displacement transducers were used with an automatic data logger to monitor the movement of the tell-tale extensometers. A precise leveling instrument was placed on a fixed platform a few meters away from the pile to measure the pile settlement. Besides, dial gauges were installed to measure the total settlement at the top of the pile under load.
Referring to the installation level of strain gage (Fig.6.8) and borehole log (Appendix B), the soil profile was divided into four layers: the backfill layer was from 0 to -7.6m, consisting of stiff clayey silt with sand and gravel. The following layer from -7.6 m to -13.6m was very soft to soft marine clay with few shell fragments. A very dense fine to coarse sand with gravel or medium strong moderately to slightly weathered sandstone was under from -13.6 m to -20.6. After that, a medium strong moderately to slightly weathered sandstone was investigated and the borehole log was terminated. The ground water table was near the ground surface.

6.2.4 Loading System and Test Procedure

The test load was applied by hydraulic jack. A kentledge was used for reaction. The detailed information on the kentledge system was not mentioned in the geotechnical reports. Therefore, a common spacing of 4D from the center of cribbage to the center of test pile and 6m × 15m cribbage area was assumed in the following calculation based on the geotechnical engineer’s experience in practice.

Two loading cycles were applied using the Quick Load Test Method for Individual Piles (ASTM D1143 Standard Test Method for Piles Under Static Axial Load). The applied load of the 2 cycles were 26400kN (2×working load, W.L. for brevity) and 39600kN (3×W.L.) respectively. In the first cycle, the loading was increased by equal loading increments of 3300kN to 26400kN; each successive load increment was held constant for 15 minutes and remained 24 hours at the maximum load of 26400kN before decreased to zero by equal unloading step of 6600kN Similarly, the second cycle was followed by equal loading/unloading step of 6600kN and applied load was remained 24 hours at the maximum load of 39600kN.
6.2.5 Back Analysis

6.2.5.1 General Settings

Due to the limitation of the current version of PLAXIS 3D Foundation, the research on the influence of the kentledge system on the load-transfer behavior and unit shaft resistance of the test pile can’t be achieved. In virtue of the PLAXIS 8, these influences can be very well studied although some assumptions may be conservative.

Therefore, the numerical simulation of this kentledge system pile load test was performed through the finite element program PLAXIS 8. The following settings were assigned and some assumptions were made:

1) Axisymmetrical model was adopted considering the boundary conditions of the pile load test.

2) The kentledge weight was taken into account by simulating load change on the cribbage while loading/unloading the test pile. 1.15 times the maximum test load capacity of dead weight of kentledge was assumed according to the experience in practice. Due to the limitation of the two-dimensional axisymmetrical model, the rectangular-shaped cribbage on both side of the test pile was simulated to a circular-shaped cribbage around it with the same width in the calculated plane. However, the distribution loads on the cribbage were taken as the realistic value applied on the rectangular-shaped cribbage for safety consideration.

3) Hardening soil model was applied to simulate the behaviour of soil layers.
4) Interface elements were incorporated along the pile to simulate the soil-pile interaction and extend 0.5 m below pile toe to reduce the corner effects causing undesirable high peaks in stress and strains around pile toe.

5) The soil was regarded as normally consolidated to slightly over-consolidated according to the laboratory consolidation test result. The OCR values of soil layers were assumed as 1.1 considering the local experience.

6) No dilatancy effects of the soil were considered.

7) Construction rate was assumed to be faster compared to consolidation rate of soils, i.e. perfectly undrained with no consolidation of the foundation soil can be assumed with little error for the soil layer such as soft marine clay and hard clayey silt.

8) The elastic compression of the pile is taken into account.

Therefore, an axisymmetrical finite element model with refined mesh consisting of 699 fifteen-node triangular elements and 5864 nodes was used and illustrated in Fig.6.9. The average element size was 0.866m$^2$. To avoid any influence of the outer boundary, the model is extended in horizontal direction to a total radius of 15m, 6m away the edge of the cribbage, and in vertical direction to 35m deep, about 13m below the pile toe.

**6.2.5.2 Material Properties**

The Young’s Modulus of test pile, $E_c$, was determined to be 24kN/mm$^2$ according to the test reports. The material properties of pile and cribbage are listed in Table 6.2.
Table 6.2 Material properties of NTUC

<table>
<thead>
<tr>
<th>Material</th>
<th>Model</th>
<th>$\gamma$ (kN/m$^3$)</th>
<th>Poisson Ratio</th>
<th>$E$ (kPa)</th>
<th>$EA$ (kN/m)</th>
<th>$EI$ (kNm$^2$/m)</th>
<th>$d$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cribbage</td>
<td>Elastic</td>
<td>20</td>
<td>0.2</td>
<td>-</td>
<td>1.60E+07</td>
<td>5.33E+04</td>
<td>0.2</td>
</tr>
<tr>
<td>Pile</td>
<td>Elastic</td>
<td>24</td>
<td>0.2</td>
<td>2.4E+07</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Four layers of soil profile were simulated in the PLAXIS model, as seen in Fig.6.9. Basic soil parameters were estimated according to the field investigation and widely accepted correlations in local practice. Equations that are introduced in Section 2.4.2.7 were used to estimate the other soil parameters of the hardening soil model. The soil properties that were adjusted according to the comparison on back-analysis result and measured data to get the best fit are listed in Table 6.3.

Table 6.3 Soil properties of NTUC

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Symbol</th>
<th>1-Backfill</th>
<th>2-Soft Marine clay</th>
<th>3-Hard clayey Silt/sandstone</th>
<th>4-Slightly Weathered Sandstone</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material model</td>
<td></td>
<td></td>
<td>H-S</td>
<td>H-S</td>
<td>H-S</td>
<td>H-S</td>
</tr>
<tr>
<td>Type of behaviour</td>
<td></td>
<td></td>
<td>Drained</td>
<td>Undrained</td>
<td>Undrained</td>
<td>Drained</td>
</tr>
<tr>
<td>Dry weight ($\gamma$)</td>
<td>kN/m$^3$</td>
<td>$\gamma_{\text{unsat}}$</td>
<td>20</td>
<td>16</td>
<td>19</td>
<td>22</td>
</tr>
<tr>
<td>Wet weight ($\gamma$)</td>
<td>kN/m$^3$</td>
<td>$\gamma_{\text{sat}}$</td>
<td>20</td>
<td>16</td>
<td>19</td>
<td>22</td>
</tr>
<tr>
<td>Young's modulus ($E_{50}$)</td>
<td>kN/m$^2$</td>
<td>$E_{\text{50}}^{ref}$</td>
<td>3.34E+04</td>
<td>6962.5</td>
<td>1.25E+05</td>
<td>4.50E+04</td>
</tr>
<tr>
<td>Oedometer modulus ($E_{\text{oed}}$)</td>
<td>kN/m$^2$</td>
<td>$E_{\text{oed}}^{ref}$</td>
<td>3.34E+04</td>
<td>7000</td>
<td>1.25E+05</td>
<td>4.50E+04</td>
</tr>
<tr>
<td>Power ($m$)</td>
<td></td>
<td></td>
<td>0.8</td>
<td>1</td>
<td>0.7</td>
<td>0.5</td>
</tr>
<tr>
<td>Unloading modulus ($E_{\text{ur}}$)</td>
<td>kN/m$^2$</td>
<td>$E_{\text{ur}}^{ref}$</td>
<td>1.00E+05</td>
<td>2.10E+04</td>
<td>3.76E+05</td>
<td>1.35E+06</td>
</tr>
<tr>
<td>Poisson's ratio ($\nu$)</td>
<td></td>
<td></td>
<td>0.20</td>
<td>0.20</td>
<td>0.20</td>
<td>0.20</td>
</tr>
<tr>
<td>Reference stress ($p$)</td>
<td>kN/m$^2$</td>
<td></td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Cohesion ($c$)</td>
<td>kN/m$^2$</td>
<td></td>
<td>10</td>
<td>15</td>
<td>220</td>
<td>600</td>
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<tr>
<td>Friction angle ($\phi$)</td>
<td>°</td>
<td></td>
<td>28</td>
<td>0</td>
<td>30</td>
<td>33</td>
</tr>
<tr>
<td>Dilatancy angle ($\psi$)</td>
<td>°</td>
<td></td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Interface strength reduction ($R_{\text{inter}}$)</td>
<td></td>
<td></td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>OCR</td>
<td></td>
<td></td>
<td>OCR 1.1</td>
<td>OCR 1.1</td>
<td>OCR 1.1</td>
<td>OCR 1.1</td>
</tr>
</tbody>
</table>

*: Do not take into consideration.

6.2.5.3 Construction Stages

The construction sequence for the pile load test was as follow:
1) Initial state of soil
2) Pile installation
3) Kentledge weight in place
4) First loading/unloading cycle (increment of 3300kN in loading procedure and 6600kN in unloading procedure)
5) Second loading/unloading cycle (increment of 6600kN in loading/unloading procedures)

Every step of loading/unloading on the test pile and cribbage was simulated in PLAXIS. For example, the distribution load on the pile was taken as $A_{\text{load}}$ and that on the cribbage was $B_{\text{load}}$; at the beginning, $A_{\text{load}}=0$ while $B_{\text{load}}=1.15 \times 39600/(2 \times 6 \times 15) = 253\text{kN/m}^2$; when pile was loaded to 3300kN, $A_{\text{load}}=3300/(3.14 \times 0.75^2) = 1867\text{kN/m}^2$, thus $B_{\text{load}}=(1.15 \times 39600-3300)/(2 \times 6 \times 15) = 234.7\text{kN/m}^2$ and so on.

6.2.6 Result and Discussion

6.2.6.1 Load-Movement Curve

The back-analyzed load-movement curves give a good match with the measured curve in Fig. 6.10. The measured movement was 7.5mm at 13200kN (1 $\times$ W.L.). At 26400kN (2 $\times$ W.L.), the movement was 24.5mm. After unloading from 2 $\times$ W.L., the residual movement was 11mm. While the calculation of FEM model gives 6.8mm at 13200kN, 20.1mm at 26400kN and 7.8mm residual movement. In the second cycle, the measured movement was 45mm at 39600kN (3 $\times$ W.L.). After unloading from
39600kN, the residual movement was 23mm. The corresponding calculated movement was 42.9mm and 23 mm respectively.

It can be seen that the Hardening-Soil model can simulate the realistic soil behavior very well. However, there are some differences without the time-dependent consolidation calculation, which may have some influence on the measurement due to the 24 hours maintenance at the maximum test load in every loading cycle during the test. Therefore, the calculated curve shows a little discrepancy at the peak load level in each cycle.

6.2.6.2 Load-Transfer Curve

The load-transfer curve of 13200kN (1×W.L.), 26400kN (2×W.L.) and 39600kN(3×W.L.) are illustrated in Figs.6.11 to 6.13, together with the corresponding measured curves. The abnormal values of the strain gauges near the ground surface on the measured curves indicate that there may be some error readings of the instruments due to some unknown reasons. Nevertheless, the calculated curves give a reasonable load-transfer trend. In the middle of the pile, the measured curves evidence a slight negative skin friction corresponding to the soft marine clay that is located at this elevation. However it is so small that it can be neglected. The curves of the calculation at this part show a vertical line indicating the characteristics of soft marine clay that it can hardly provide any shaft resistance. At the lower part, two layers of load-transfer curves show a very close match. It is concluded that the estimated soil properties of the slightly weathered sandstone layer are reasonable enough although it is difficult to obtain the soil properties of rock-like socket through laboratory experiment.
The mobilized end bearing was 2080kN at the test load of 13200kN, 5300kN at the test load of 26400kN and 10250kN at the test load of 39600kN according to the test report. Similarly, the corresponding results of the calculation are 2680kN, 6570kN and 12860kN.

6.2.6.3 Unit Shaft Resistance Curve

Similarly, the unit shaft resistance curve of 13200kN (1×W.L.), 26400kN (2×W.L.) and 39600kN (3×W.L.) are illustrated in Figs.6.14 to 6.16. together with the corresponding measured curves. Both of the results are reasonable and agree very well in the practical sense.

Due to the error in readings of instruments, the measured axial forces of the pile at the first soil layer are much higher than the actual load on the pile head, which is unreasonable. When some adjustment is made to omit those error readings at the level A, the ultimate unit shaft resistance would be about 100kN/m², quite close to the calculated value of about 90kN/m². There is a soft marine clay layer at the depth of EL.-7.6 to EL.-13.6, which correspond to the unit shaft resistance value of near zero in both measured and calculated curves. Furthermore, the ultimate unit shaft resistance values of the third and fourth soil layers are calculated to be 411kN/m² and 2050kN/m², which are very close to the measured values of 434.4kN/m² and 2010.1kN/m².
6.2.7 Evaluation of Kentledge Influence

To evaluate the influence of the kentledge system on the measurement of the pile load test in practice, a re-calculation of the same FEM model was made without considering the influence of kentledge weight, which is the method that a geotechnical engineer usually used in practice. The material properties remained the same and all the loading/unloading steps were followed except the kentledge weight was kept zero at any construction stage. The results are compared with the preceding case.

6.2.7.1 Load-Movement Curve

The load-movement curve without kentledge is compared with the curve with kentledge, together with the measured curve, as in Fig.6.17. It can be seen that the load-movement behavior with kentledge is a little bit stiffer than that without kentledge. At the loads of 13200kN, 26400kN and 39600kN, the movements of pile head without considering the kentledge effect are 10.3mm, 26.8mm, 51.7mm respectively. Compared with those considering the kentledge effect, 6.8mm, 20.1mm, 42.9mm, the $F_c$ values are 1.51, 1.33 and 1.21 respectively. It further proves that the influence of the kentledge is significant at the commonly used spacing of 3~5D (diameter of the pile) between the center of cribbage and the center of test pile.

6.2.7.2 Load-Transfer Curve

The load-transfer curves with and without kentledge at the three different working load levels are illustrated in Fig.6.18 to Fig.6.20. It seems that the influence of the kentledge becomes less as the test load level increases. This may be because the weight of kentledge decreases while the load applied on the test pile increases and the level of
the kentledge weight determines the degree of the interaction effect between the pile and surrounding soil.

The discrepancy between the load-transfer curves with and without kentledge is becoming less along the depth. The upper soil layers bear more loads in the calculation with kentledge than those without kentledge. It is suggested that the pile-soil interaction become less along the depth.

6.2.7.3 Unit Shaft Resistance Curve

Similarly, the unit shaft resistance curves with and without kentledge at the three different working load levels are shown in Fig.6.21 to Fig.6.23. It is clear that the unit shaft resistance at the upper part of the test pile increases in the kentledge case than in the other. And this significance becomes less when the working load level increases. The unit shaft resistance at the lower part remains the same for both cases.

The total shaft resistance $Q_s$ increases under the influence of the kentledge weight, thus the end bearing resistance $Q_b$ becomes less in the kentledge case than in the other under the same working load. This can be evidenced in Fig. 6.24.

6.2.8 Conclusions

Through the back-analysis of the above case history with FEM, some findings are further illustrated:

1) It is suggested that the influence of the reaction system on the settlement of the test pile is usually under-estimated in practice. The commonly recommended
minimum spacing of 3~5D (diameter of the pile) between test pile and reaction system may not be enough, as it tends to have greater influence on test pile results than desired.

2) The kentledge weight increases the pile-soil interaction by increasing the unit shaft resistance of the pile without kentledge near the ground surface. As a result, the end bearing becomes less in the kentledge case than in the other under the same load.

3) The influence of kentledge becomes less as the load applied on the test pile increases, as well as at greater along the depth.
Fig. 6.1  Pile Load Arrangement and Design Soil Profile
Fig. 6.2 3D FEM Model with Four Reaction Piles
Fig. 6.3  Load-Settlement Curve of 4 Reaction Piles System

Fig. 6.4  Comparison of Load-Settlement Curve of 4 Reaction Piles System with Single Pile
Fig. 6.5  3D FEM Model with Two Reaction Piles
Fig. 6.6  Comparison of Load-Settlement Curve of 4 Reaction Piles System with 2 Reaction Piles

Fig. 6.7  Influence of Different Numbers of Reaction Piles
Fig. 6.8  Location of Instruments in Test Pile of NTUC
Cribbage
Backfill
Soft Marine Clay
Hard Clayey Silty with Sand
Slightly Weathered Sandstone

Fig. 6.9  FEM Model of NTUC


**Fig. 6.10**  Load-Movement Curve

**Fig. 6.11**  Load-Transfer Curve at $1 \times W.L.$
Fig. 6.12  Load-Transfer Curve at 2×W.L.

Fig. 6.13  Load-Transfer Curve at 3×W.L.
Fig. 6.14  Unit Shaft Resistance Curve at 1×W.L.

Fig. 6.15  Unit Shaft Resistance Curve at 2×W.L.
Fig. 6.16  Unit Shaft Resistance Curve at 3 × W.L.
Fig. 6.17  Comparison of Load-Movement Curve
Fig. 6.18  Comparison of Load-Transfer Curve at $1 \times W.L.$

Fig. 6.19  Comparison of Load-Transfer Curve at $2 \times W.L.$
Fig. 6.20  Comparison of Load-Transfer Curve at 3 × W.L.

Fig. 6.21  Comparison of Unit Shaft Resistance Curve at 1 × W.L.
Fig. 6.22  Comparison of Unit Shaft Resistance Curve at 2×W.L.

Fig. 6.23  Comparison of Unit Shaft Resistance Curve at 3×W.L.
Fig. 6.24  Comparison of Shaft and End Bearing Resistance vs. Movement curve
7.1 Conclusions

3-dimentional FEM study on the influence of different reaction system (kentledge system and reaction tension pile system) on the results of conventional static load test and 2-dimentional FEM study on Osterberg-cell test and comparison of these two kinds of pile load tests were performed under simplified soil profile and boundary conditions. The same research methods were applied to some case histories to affirm findings. The findings in this thesis are:

7.1.1 Influence of Reaction System on Conventional Pile Load Test

1) The numerical results indicate that the influence of the reaction system on the settlement of the test pile is usually under-estimated in practice. The commonly recommended minimum spacing of 3~5D (diameter of the pile) between test pile and reaction system may not be adequate, as it tends to have greater influence on test pile results than desired.

2) For both reaction system, the larger the L (length of the pile)/D (diameter of the pile) ratio, the stronger the pile-soil interaction, thus the higher the correction factor needed for test interpretation; the softer the soil compared to the pile stiffness (the larger the K), the less the influence of the reaction system on the test results.
3) Kentledge system may have larger influence on the test pile compared to the reaction pile system under the same test condition due to the additional weight higher than load capacity of the test pile needed for safety consideration.

4) For the kentledge system, the influence of the width of cribbage is significant for spacing less than 5D; the narrower the width of cribbage, the less the influence on the test results. The influence of the area of cribbage can not be neglected for spacing less than 4D; the larger area of the cribbage has the advantage of reducing the influence on the test pile even at small spacing.

5) The kentledge weight increases the pile-soil interaction by increasing the unit shaft resistance of the pile. As a result, the end bearing becomes less in the kentledge case than in the case without kentledge under the same load.

6) The influence of kentledge becomes less as the load applied on the test pile increases, as well as along the greater depth of the pile.

7) For the reaction pile system, the diameter of pile D does not influence the pile-soil-pile interaction for the same spacing ratio of diameter.

8) For the reaction pile system, at lower load levels, the soil behaviour is more like elastic, the soil-pile interaction can be regarded as independent of the stress state of the soil; however, the higher load level causes more nonlinearity and soil-pile interaction is less around the test pile.

9) For the reaction pile system, the interaction effect would be greater as the number of the reaction piles increases.

7.1.2  Comparison of Osterberg-Cell Load Test with Conventional Load Test
1) It is concluded from the preceding FEM study that O-cell test result can provide not only the same soil-pile interaction information as conventional head-down static loading test, but also allow for separate determination of the shaft resistance and end bearing components.

2) The FEM computation indicates that the shaft resistance-movement curve for upward movement of the pile is fairly comparable with the downward shaft resistance-movement component of a conventional head-down test. The result of undrained analysis suggests that some difference of mobilized shaft resistance may exist between O-cell test and conventional test in the short-term view due to the different sets of excess pore pressures generated. However, this discrepancy will attenuate in the long-term.

3) Also the end bearing load-movement curve obtained from an O-cell test is quite similar to the end bearing-load movement component curve of a conventional head-down test. The curve of O-cell test may reflect a stiffer load-movement response than that of conventional test probably due to the shorter load transfer distance between pile toe and point of loading application, so that the end bearing of the O-cell test is mobilized much earlier compared to conventional test.

4) The equivalent head down load-movement curve of the O-cell test simulated by PLAXIS 8 gives a little stiffer load-movement response and slightly higher ultimate capacity than those of conventional test. Apart from the computational schemes of the FEM program itself, the differences of effective stress around the pile due to the different excess pore pressures generated from the different load-
transfer mechanism among these kinds of pile load tests contributed to the
differences of unit shaft resistance of these test piles under the same pile
movement. When the drained analyses were made and long-term soil-pile
interaction was considered, both the O-cell test and conventional test gave very
close results.

5) The elastic compression of the pile could be considered for an O-cell test so that
the more agreeable results with that of conventional test can be obtained. However,
the triangle method recommended in this thesis will give a slightly unconservative
estimate.

6) The O-cell level has essentially no influence on the test results. In this thesis, the
FEM analysis indicates that bottom O-cell test and middle O-cell test would give
the same results. In practice, the O-cell level should be determined based on
adequate resistance from the lower portion of the pile (end bearing plus shaft
resistance of lower portion) to counteract the upper resistance of the pile.

7.2 Recommendations for Further Research

During the composition period of this thesis, the PLAXIS 3D Foundation was just
releasing its beta to the first version, which led to a lot of difficulty for the author who
was trying to carry out his research with a view of 3-dimension due to the limitation of
the program function. Although the program was applied and some conclusions were
achieved, some simulations were not done much better. In the course of study, several
areas have shown potential for making it better or further research.
The influence of reaction system on the load transfer behavior, unit shaft resistance and load capacity of the test pile can be studied with the more advanced version of 3D FEM code. In addition, other types of reaction system such as anchors or inclined reaction piles can be examined.

Effects of consolidation of more permeable soils with close drainage boundaries on pile load test results can also be studied using FEM method.
REFERENCES


BS8004 (1986), British Standard Code of Practice for foundations.


APPENDIX A

Bored Hole Log for Gopeng Street Project

<table>
<thead>
<tr>
<th>DEPTH</th>
<th>SAMPLE NO.</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5 m</td>
<td>S-1</td>
<td>Medium stiff, light purple to grey Clayey Silt.</td>
</tr>
<tr>
<td>3.0 m</td>
<td>S-2</td>
<td>Loose reddish brown Clayey fine Sand.</td>
</tr>
<tr>
<td>6.5 m</td>
<td>S-3</td>
<td>Very dense, yellowish brown to whitish grey Clayey fine Sand.</td>
</tr>
<tr>
<td>12.5 m</td>
<td>S-7</td>
<td>Very dense, grey Silty Sand with Siltstone fragments.</td>
</tr>
</tbody>
</table>

MOH AND ASSOCIATES Consulting Engineers

SUBSURFACE EXPLORATION LOG

PROJECT: Proposed Residential Development of Gopeng Street

BORED BY: Geotechnical Instrumentation Services

Prepared by: Checked by: Date: 05/08/03
<table>
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<tr>
<th>M</th>
<th>SAMPLE NO.</th>
<th>DESCRIPTION</th>
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</thead>
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<tr>
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<td>C-1</td>
<td>Moderately weak, grey, moderately to highly fractured slightly weathered SILTSTONE with completely decomposed SILTSTONE of 21.00m to 21.40m</td>
</tr>
<tr>
<td>22</td>
<td>C-2</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>C-3</td>
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<tr>
<td></td>
<td>C-6</td>
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</tr>
<tr>
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<td>C-7</td>
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</tr>
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<td></td>
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<td>C-10</td>
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End of Boring

---

MOH AND ASSOCIATES
Consulting Engineers

PROJECT: Proposed Residential Development at Gopeng Street

SUBSURFACE EXPLORATION LOG

BORED BY: Geotechnical Instrumentation Services

Prepared by: [Signature] Checked by: [Signature] Date: 05/01/03

159
# APPENDIX B

## Bored Hole Log for NTUC Project

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Soil Profile</th>
<th>SPT N-Value (Blows)</th>
<th>Soil Test Reference</th>
<th>Shear Strength (kPa)</th>
<th>Moisture Content (%)</th>
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<td>Ref</td>
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<td>25</td>
</tr>
<tr>
<td>2</td>
<td>0.03-0.05m</td>
<td>15</td>
<td>Ref</td>
<td>70</td>
<td>25</td>
</tr>
<tr>
<td>3</td>
<td>0.05-0.1m</td>
<td>20</td>
<td>Ref</td>
<td>100</td>
<td>50</td>
</tr>
<tr>
<td>4</td>
<td>0.10-0.2m</td>
<td>25</td>
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<td></td>
<td></td>
</tr>
<tr>
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<td>0.60-0.7m</td>
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<td>1.20-1.3m</td>
<td></td>
<td>Ref</td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>1.30-1.4m</td>
<td></td>
<td>Ref</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>1.40-1.5m</td>
<td></td>
<td>Ref</td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>1.50-1.6m</td>
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<td>Ref</td>
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</tr>
<tr>
<td>19</td>
<td>1.60-1.7m</td>
<td></td>
<td>Ref</td>
<td></td>
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<tr>
<td>20</td>
<td>1.70-1.8m</td>
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<td>Ref</td>
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</table>

**Shear Strength Tests**
- Undrained triaxial (US)
- Undrained triaxial (OS)
- Consolated undrained triaxial (DU)
- Consolated drained triaxial (DS)
- Laboratory vane
- Field vane

**Sample Type**
- Undisturbed
- SPT N-Value

**Additional Information**

**Client:** SINGAPORE LABOUR FOUNDATION

**Project:** S.I. WORKS & UNDERGROUND SERVICES DETECTION WORKS FOR PROPOSED NTUC BUILDING DEVELOPMENT AT Raffles Quay/Marina Boulevard

**User:** ZAW

**Date:** 27/2-7/3/01

**ECON LAB:** 29407.897 m

**Field Vane Test:**
- Type: ROTARY
- Bore (m): 100
- Reduced level (m): 102.530

**References:**

**Signatures:**
- Drawn By: MAY
- Checked By: ZAW
<table>
<thead>
<tr>
<th>Soil Type</th>
<th>SPT N-Value</th>
<th>Blaine Value</th>
<th>Moisture Content</th>
<th>Shear Strength</th>
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</thead>
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<tr>
<td>CL</td>
<td>65</td>
<td>200</td>
<td>8%</td>
<td>100</td>
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<tr>
<td>CH</td>
<td>85</td>
<td>150</td>
<td>12%</td>
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<tr>
<td>CL</td>
<td>70</td>
<td>180</td>
<td>10%</td>
<td>110</td>
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</table>

**Sample Reference**

- **Sample Type**: Undisturbed
- **Sample Description**: Soil profile
- **Sample Reference**: 27/7/301

**Shear Strength**

- **Peak Shear Stress (kPa)**: 100
- **Bulk Density (kN/m³)**: 190

**Notes**

- Soil profile and sample information provided for reference and analysis.
- Additional geological and engineering data available for evaluation.

**Contact**

- Econ Singapore Building Development Services
- For further information, please contact info@econ-sg.com