The Behaviour of Axially Loaded Piles Subjected to Lateral Soil Movements

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Dedicated to my teachers, my parents, my wife and my Lord.
The fear of the LORD is the instruction of wisdom; and before honor is humility.

Proverbs 15:33
TABLE OF CONTENTS

Abstract xv
Acknowledgement xvii
Declaration xix
List of publications xx
Notations xxi

Chapter 1: Introduction
1.0 Introduction ................................................................. 1
1.1 Aim and objectives ...................................................... 3
1.2 Thesis outline ............................................................ 4

Chapter 2: Literature review
2.0 Introduction ............................................................... 6
2.1 Analysis methods .......................................................... 6
2.1.1 Pressure based methods ............................................ 6
2.1.2 Displacement based methods .................................... 13
2.1.3 Finite element methods ............................................ 14
2.2 Laboratory model tests on piles subjected to lateral soil movements .... 18
2.2.1 Published laboratory model pile tests ......................... 19
2.2.1 Summary of the laboratory model tests ..................... 29
2.3 Pile subjected to combined loadings .............................. 31
2.4 Conclusions .............................................................. 34

Chapter 3: Experimental investigation
3.0 Introduction ............................................................... 35
3.1 Experimental investigation ............................................ 36
3.2 Apparatus design ........................................................ 36
3.2.1 Laminar shear box .................................................. 37
3.2.2 Investigation of sand displacement in the shear box ....... 47
3.2.3 Load application equipment .................................... 53
3.2.3.1 Pile driving with the vertical jacking system ............ 54
3.2.3.2 Application of axial load ....................................... 55
3.2.3.3 The horizontal soil movement system .................. 56
3.2.4 Sand properties and preparation ............................... 57
3.2.5 Model pile .......................................................... 60
3.2.6 Data acquisition system ......................................... 62
3.3 Experimental procedure ............................................ 64
3.3.1 Data analysis and processing .................................... 67
3.3.1.1 Numerical differentiation ..................................... 68
3.3.1.2 Numerical integration .......................................... 70
3.4 Limitations and advantages of small-scale model tests ............. 71
3.5 Intended use of model test results in this thesis .................. 72
3.6 Recommendations for future work and use of model test results .......... 72
3.7 Summary ............................................................................................ 72

Chapter 4: Single pile tests

4.0 Introduction of single pile tests ............................................................ 74
4.1 Sign convention .................................................................................. 76
4.2 Tests for single piles subjected to rectangular soil movement profile .... 77
4.2.1 Boundary effect on model test ......................................................... 80
4.2.2 Experimental results with $L_m/L_s = 200/500$ ................................. 84
4.2.2.1 Density effect on 32 mm diameter pile ......................................... 86
4.2.2.2 Influence of pile diameter ............................................................ 87
4.2.2.3 Influence of axial load on 32 mm diameter pile ............................ 88
4.2.2.4 Influence of axial load on 50 mm diameter pile ............................ 89
4.2.3 Experimental results with $L_m/L_s = 400/300$ ................................. 91
4.2.3.1 Density effect of 50 mm diameter pile ........................................... 93
4.2.3.2 Influence of pile diameter ............................................................ 94
4.2.3.3 Influence of axial load on 32 mm diameter pile ......................... 95
4.2.3.4 Influence of axial load on 50 mm diameter pile ......................... 95
4.2.4 Summary of single piles subjected to rectangular soil movement ... 97
4.2.4.1 The relationship between the $M_{\text{max}}$ and the pile diameter at different $L_m/L_s$ ratios ....................................................... 97
4.2.4.2 The relationship between $M_{\text{max}}$ and axial load at different ratios of $L_m/L_s$ ................................................................. 99
4.2.4.3 The relationship between pile deflection and axial load at different ratios of $L_m/L_s$ ................................................................. 100
4.3 Tests for single piles subjected to triangular soil movement profiles .... 101
4.3.1 Experimental results with $L_m/L_s = 200/500$ ................................. 102
4.3.1.1 Influence of pile diameter ............................................................ 102
4.3.1.2 Influence of axial load ................................................................. 104
4.3.2 Experimental results with $L_m/L_s = 400/300$ ................................. 107
4.3.2.1 Influence of pile diameter ............................................................ 107
4.3.2.2 Influence of axial load ................................................................. 108
4.4 Tests for single piles subjected to arc soil movement profiles .......... 110
4.4.1 Experimental results with $L_m/L_s = 200/500$ ................................. 111
4.4.1.1 Influence of pile diameter ............................................................ 111
4.4.1.2 Influence of axial load ................................................................. 113
4.4.2 Experimental results with $L_m/L_s = 400/300$ ................................. 117
4.4.2.1 Influence of pile diameter ............................................................ 117
4.4.2.2 Influence of axial load ................................................................. 118
4.5 Limiting pressure obtained from model tests ...................................... 120
4.6 Comparison of pile responses obtained from different soil movement profiles ................................................................. 122
4.6.1 Tests with $L_m/L_s = 400/300$ .......................................................... 122
4.6.2 Tests with $L_m/L_s = 200/500$ .......................................................... 126
4.6.3 On general $M_{\text{max}}$ against $S_{\text{max}}$ relationship ............................ 129
4.7 Limitations and possible improvements on model tests ................... 130
4.8 Conclusions ...................................................................................... 131
Chapter 5: Pile group tests

5.0 Introduction ........................................................................................................... 134
5.1 Tests for pile groups subjected to rectangular soil movement profile .......... 136
5.2 Setup of pile group testing ................................................................................... 138
5.2.1 Pile arrangement and instrument locations ......................................................... 138
5.2.2 Pile spacings and pile cap details ....................................................................... 139
5.2.3 Pile installation and axial load application ............................................................. 141
5.2.4 Sign convention and interpretation of results ....................................................... 141
5.2.5 Boundary conditions and limitation of shear box ............................................... 141
5.3 Results for piles in a row (1 × 2) ............................................................................ 143
5.3.1 The effect of axial load with Lm/Ls of 200/500 .................................................... 143
5.3.2 The effect of axial load with Lm/Ls of 400/300 .................................................... 147
5.4 Results of piles in a line (2 × 1) ............................................................................... 150
5.4.1 The effect of axial load with Lm/Ls of 200/500 .................................................... 150
5.4.2 The effect of axial load with Lm/Ls of 400/300 .................................................... 152
5.5 Results of piles in square arrangement (2 × 2) ...................................................... 155
5.5.1 Test on 32 mm diameter piles with Lm/Ls of 200/500 ........................................... 155
5.5.1.1 Piles in 3d spacing ..................................................................................... 155
5.5.1.2 Piles in 5d spacing ..................................................................................... 158
5.5.2 Test on 32 mm diameter piles with Lm/Ls of 400/300 ........................................... 160
5.5.2.1 Piles in 3d spacing ..................................................................................... 160
5.5.2.2 Piles in 5d spacing ..................................................................................... 162
5.5.3 Test on 50 mm diameter piles with Lm/Ls of 200/500 ........................................... 164
5.5.3.1 Piles in 3d spacing ..................................................................................... 164
5.5.3.2 Piles in 5d spacing ..................................................................................... 166
5.5.4 Summary of common trends noted on the 2 × 2 pile groups ......................... 168
5.6 The effect of Lm/Ls ratio ....................................................................................... 169
5.6.1 The effect of Lm/Ls on piles in a row (1 × 2) ......................................................... 169
5.6.2 The effect of Lm/Ls on piles in a line (2 × 1) ......................................................... 172
5.6.3 The effect of Lm/Ls on piles in a square (2 × 2) ..................................................... 173
5.7 The effect of pile diameter ..................................................................................... 175
5.8 The effect of pile spacing ....................................................................................... 177
5.8.1 Effect of spacing on piles in a row (1 × 2, Lm/Ls = 200/500, d = 32 mm) ......... 177
5.8.2 Effect of spacing on piles in a square (2 × 2, Lm/Ls = 200/500, d = 32 mm) .......... 179
5.8.3 Effect of spacing on piles in a square (2 × 2, Lm/Ls = 200/500, d = 50 mm) ......... 183
5.8.4 Effect of spacing on piles in a square (2 × 2, Lm/Ls = 400/300, d = 32 mm) ......... 187
5.8.5 Summary on the effect of pile spacing ................................................................. 189
5.9 The effect of pile group arrangement ...................................................................... 191
5.10 Limiting soil pressure profile ............................................................................. 195
5.10.1 Maximum soil pressure in the moving layer (Lm) ............................................... 196
5.10.2 Maximum soil pressure in the stable layer (Ls) ................................................... 198
5.11 On Mmax against Smax relationship on the pile group tests ................................. 201
5.12 Other findings ..................................................................................................... 203
5.12.1 Lateral force to cause soil movement ................................................................. 203
5.12.2 Pile penetration resistance ............................................................................... 205
5.13 Limitation on the pile group tests ...................................................................... 206
Chapter 6: Three dimensional finite difference analysis

6.0 Introduction .................................................. 210
6.1 Program details .............................................. 210
6.2 Standard model setup ....................................... 210
6.2.1 Geometry and mesh ..................................... 211
6.2.2 Sand properties .......................................... 212
6.2.3 Mohr-Coulomb constituitive model for sand .............. 212
6.2.4 Interface model ........................................... 212
6.2.5 Model pile .................................................. 213
6.2.5.1 Calibration of model piles .............................. 214
6.2.6 Summary of the input parameters for the standard model ..... 215
6.3 Application of stress and movements .................. 215
6.4 Comparative study on the standard model ............... 216
6.4.1 Effect of mesh density ................................... 216
6.4.2 Effect of constitutive models .......................... 218
6.4.3 Parametric study ........................................... 219
6.4.3.1 Effect of Young's modulus ......................... 220
6.4.3.2 Effect of dilation angle of sand ....................... 221
6.4.3.3 Effect of interface element .......................... 222
6.4.4 Summary of the parametric study .................... 224
6.5 FLAC3D analyses performed on model tests ............ 225
6.5.1 Test on 50 mm pile at $L_m/L_s = 400/300$ and $Q = 0$ N ........ 226
6.5.2 Test on 50 mm pile at $L_m/L_s = 400/300$ and $Q = 294$ N .......... 231
6.5.3 Test on 50 mm pile at $L_m/L_s = 400/300$ and $Q = 589$ N .......... 233
6.5.4 Test on 32 mm pile at $L_m/L_s = 200/500$ and $Q = 0$ N ............ 237
6.5.5 Test on 50 mm pile at $L_m/L_s = 200/500$ and $Q = 0$ N ............ 240
6.6 The effect of $L_m/L_s$ on the pile deflection mode .......... 245
6.7 The effect of $L_m/L_s$ on the pile response ................. 247
6.7.1 The effect of $L_m/L_s$ on the pile response with $Q = 0$ N ............ 247
6.7.2 The effect of $L_m/L_s$ on the pile response with $Q = 294$ N ........... 253
6.7.3 The effect of sand dilation angle on the pile response different Lm/Ls ratios ...................................................... 259
6.8 On $M_{\text{max}}$ against $S_{\text{max}}$ relationship obtained from the numerical analysis .262
6.9 Comparison between prediction of FLAC3D and an existing solution ...... 264
6.9.1 Chen and Poulos (1997) .................................. 264
6.10 Limitation and possible improvement on the FLAC3D model .......... 266
6.11 Conclusions .................................................. 267

Chapter 7: Single pile and pile group tests in clay

7.0 Introduction .................................................. 269
7.1 Test description .............................................. 269
7.1.1 Shear box ................................................. 269
7.1.2 Clay properties and preparation procedure ............... 272
7.1.3 Model pile .................................................. 273
7.1.4 Details of the tests performed .......................... 274
7.2 Determination of undrained shear strength .................. 276
7.3 Single pile tests .............................................. 277
7.3.1 Test C_2-5_25_170805 ................................................................. 277
7.3.2 Test C_2-5_32_250605 ................................................................. 277
7.3.3 Test C_4-3_32_290705 ................................................................. 278
7.3.4 Summary of the single pile tests .................................................. 282
7.4 Pile group tests ............................................................................. 283
7.4.1 Test C_2-5_32_1×2_3d_230705 .................................................... 283
7.4.2 Test C_2-5_32_1×2_5d_300705 .................................................... 286
7.4.3 Test C_2-5_32_2×1_3d_270705 .................................................... 287
7.4.4 The effect of pile group arrangements ......................................... 292
7.5 Observation of soil plugging ............................................................. 293
7.6 Limitation of current testing with clay ............................................. 294
7.7 Conclusions .................................................................................. 295

Chapter 8: Simplified solution to analyse rigid piles subjected to lateral soil movement

8.0 Introduction .................................................................................. 297
8.1 Limiting pressure profile for sand .................................................. 297
8.2 Soil movement to mobilise limiting pressure – with reference to pipe depth .......................................................... 298
8.3 Soil movement to mobilise limiting pressure with reference to pile diameter .................................................. 300
8.4 Three different failure modes of the rigid pile .................................. 301
8.5 Estimation of pressure profile in L_m (passive section) ...................... 302
8.6 Solution of pressure acting in L_s (active section) ............................. 303
8.7 Pile rigidity .................................................................................. 304
8.8 Solution flowchart ....................................................................... 305
8.9 Prediction on current single pile tests .............................................. 305
8.10 Case studies ................................................................................ 310
8.10.1 Cases 1 and 2 (Chen, 1994) ....................................................... 310
8.10.2 Case 3 (Tsuchiya et al, 2001) ..................................................... 314
8.10.3 Case 4 (Kalteziotis et al, 1993) .................................................. 317
8.11 Limitation of current solution ........................................................ 321
8.12 Conclusions ................................................................................ 322

Chapter 9: Conclusions and recommendations for future research work

9.0 Introduction .................................................................................. 323
9.1 Experimental study: piles in sand ................................................... 323
9.1.1 Single pile tests ......................................................................... 323
9.1.2 Pile group tests ......................................................................... 324
9.2 Numerical analysis: single piles in sand ........................................ 325
9.3 Experimental study: piles in clay ................................................... 326
9.3.1 Single pile tests ......................................................................... 326
9.3.2 Pile group tests ......................................................................... 327
9.4 Simplified solution to analyse rigid pile subjected to lateral soil movement .......................................................... 327
9.5 Recommendations for future research work .................................. 328
9.5.1 Experimental apparatus ............................................................. 328
9.5.2 Single pile tests ......................................................................... 329
9.5.3 Pile group tests ......................................................................... 329
9.5.4 Numerical analysis.................................................................329
9.5.5 Simplified solution.................................................................330
9.6 Concluding remarks.................................................................330

References.........................................................................................331

Appendices
Appendix A: Sand properties..............................................................344
Appendix B: Single pile test results......................................................348
Appendix C: Pile group test results.......................................................387
Appendix D: Standard model input data file for FLAC$^{3D}$........................429
LIST OF FIGURES

Figure 2.1 Begemann and De Leeuw’s design method (after Steward, 1992) ........... 8
Figure 2.2 De Beer and Wallays’ method for $F_s > 1.6$ (after Steward, 1992) ........ 9
Figure 2.3 De Beer and Wallays’ method for $F_s < 1.6$ (after Steward, 1992) ...... 9
Figure 2.4 Tschebotarioff’s design method (after Steward, 1992) ..................... 10
Figure 2.5 Stabilising piles in a row through plastically deforming ground ....... 11
Figure 2.6 Method of Springman (1989) (after Steward, 1992) ......................... 12
Figure 2.7 Model test (after Fukumoto, 1975) .................................................. 19
Figure 2.8 Model setup (Matsui et al, 1982) ...................................................... 22
Figure 2.9 Arrangement of centrifuge model (Springman, 1989) ......................... 23
Figure 2.10 Sectional view of model (Anagnostopoulos et al, 1991) ............... 23
Figure 2.11 Overview of experimental apparatus (Chen, 1994) ......................... 25
Figure 2.12 Configuration of centrifuge model (Stewart, 1992) ....................... 26
Figure 2.13 Centrifuge test model (Bransby, 1995) ........................................... 26
Figure 2.14 Model test setup (Pan, 1998) .......................................................... 27
Figure 2.15 Configuration of centrifuge model (Leung, 2000) ......................... 27
Figure 2.16 Large-scale laminar shear box design (Tsuchiya et al, 2001) ....... 28
Figure 2.17 Similarity between axially loaded piles subjected to lateral load and lateral soil movement .......................................................... 32
Figure 3.1 Presentation chart of experimental work in this thesis ....................... 35
Figure 3.2 Experimental apparatus setup ......................................................... 39
Figure 3.3 Mechanism of triangular soil movement profile ......................... 40
Figure 3.4 Application of triangular soil movement profile with $L_m/L_s = 200/500$ at $w_s$ of 10 mm, 20 mm, 70 mm and 150 mm ............. 42
Figure 3.5 Mechanism of arc soil movement profile .................................... 44
Figure 3.6 Application of arc soil movement profile with $L_m/L_s = 200/500$ $w_s$ of 10 mm to 120 mm, with 10 mm incrementals .......... 46
Figure 3.7 The sand displacement at the surface of the shear box at $w_s$ of 140 mm 48
Figure 3.8 Typical sand displacement contour at the surface of the shear box .... 48
Figure 3.9 Initial locations of coloured sand placed within the shear box ......... 49
Figure 3.10 Coloured sand recovered at approximately 100 mm depth below the surface and measured ...................................................... 49
Figure 3.11 Some indication of the extent of shear zone within the shear box for tests with $L_m/L_s = 200/500$ mm and at $w_s > 60$ mm ....................... 52
Figure 3.12 Load reaction frame .................................................................... 53
Figure 3.13 Vertical hydraulic jack and connection to a model pile ............... 54
Figure 3.14 Axial load applied on the pile head by placing weight blocks of 10 kg each (98.1 N) ............................................................... 55
Figure 3.15 Loading blocks to simulate different movement profiles .......... 56
Figure 3.16 Dimensions of the loading blocks to simulate different soil movement profiles ............................................................. 56
Figure 3.17 Particle size distribution ................................................................. 57
Figure 3.18 Sand rainer used for sand preparation ........................................ 59
Figure 3.19 The relationship between dry density and height of fall ............. 59
Figure 3.20 Instrumented model pile ............................................................ 61
Figure 3.21 Data acquisition control panel .................................................... 63
Figure 3.22 Measuring the displacement of the laminar frame ................. 64
Figure 3.23 Derivation of other pile responses from the measured bending vii
moment profile........................................................................................................68
Figure 3.24 The locations of the stain gauges on the pile...........................................68
Figure 3.25 Derivation of the pile rotation and the pile deflection from the measured bending moment ..........................70
Figure 4.1 Flowchart showing presentation of single pile tests in this chapter ............75
Figure 4.2 Single pile sign convention ......................................................................76
Figure 4.3 Test identification number for single piles in sand ....................................77
Figure 4.4 Location of a model pile in the shear box ..................................................80
Figure 4.5 Responses of the piles tested at different locations in the shear box
with Lm/Ls = 200/500: a) bending moment profiles; and b) soil reaction profiles ..................................................82
Figure 4.6 Responses of the piles tested at different locations in the shear box
with Lm/Ls = 400/300: a) bending moment profiles; and b) soil reaction profiles ..................................................82
Figure 4.7 The response of the 32 mm diameter pile at Lm/Ls = 200/500 .................85
Figure 4.8 Results from 32 mm pile tested on different densities: a) bending
moment profiles; b) Mmax vs. density; and c) Mmax vs. relative density ...............87
Figure 4.9 Responses of 25 mm, 32 mm and 50 mm diameter piles at
Ls/Lm = 200/500: a) bending moment profile; and b) Mmax against ws .............88
Figure 4.10 Responses of 32 mm diameter pile with axial loads applied on the
top head................................................................................................................89
Figure 4.11 Responses of 50 mm diameter pile with different axial loads applied on
the pile head at soil movement of: a) w = 20 mm; b) w = 60 mm; and c) Mmax against w..........................................................90
Figure 4.12 The response of the 32 mm diameter pile at Lm/Ls = 400/300 ............92
Figure 4.13 Responses of 50 mm diameter pile tested at two densities: b) pile
rotation profiles; and c) pile deflection profiles ..............................................94
Figure 4.14 Bending moment profiles of 25 mm, 32 mm and 50 mm diameter piles
(Ls/Lm = 400/300) .........................................................................................94
Figure 4.15 Responses of 32 mm diameter pile tested at three different axial loads
(0 N, 294 N and 589 N) ..................................................................................95
Figure 4.16 Responses of 50 mm diameter pile tested at three different axial loads
(0 N, 294 N and 589 N) ..................................................................................96
Figure 4.17 Mmax against pile diameter for single pile tests conducted at different
ratios of Lm/Ls ..............................................................................................98
Figure 4.18 Mmax against axial load for single pile tests conducted at different
ratios of Lm/Ls ............................................................................................99
Figure 4.19 Mmax against axial load for single pile tests conducted at different
ratios of Lm/Ls ..........................................................................................100
Figure 4.20 Responses of 32 mm and 50 mm diameter piles subjected to triangular
profile of soil movement: a) bending moment profile; b) Mmax against w; c) pile deflection profile ...........................................103
Figure 4.21 Responses of 32 mm diameter pile subjected to different axial loads and
triangular soil movement profile: a) at w = 60 mm; b) at w = 90 mm; and c) Mmax against w ......................................................105
Figure 4.22 Responses of 50 mm diameter pile subjected to different axial loads and
triangular soil movement profile ...................................................................106
Figure 4.23 Responses of 32 mm and 50 mm diameter piles subjected to triangular
soil movement profile: a) bending moment profile; and b) pile deflection profile ...........................................................................107
Figure 4.24 Responses of 32 mm diameter pile subjected to different axial loads (0 N and 294 N) ............................................................................................. 108
Figure 4.25 Responses of 50 mm diameter pile subjected to different axial loads (0 N and 294 N) ............................................................................................. 109
Figure 4.26 Responses of 32 mm and 50 mm diameter piles subjected to triangular soil movement profile: a) bending moment profile; and b) $M_{\text{max}}$ against $w_s$. 112
Figure 4.27 Responses of 32 mm diameter pile subjected to different axial loads and arc soil movement profile........................................................................ 114
Figure 4.28 Responses of 50 mm diameter pile subjected to different axial loads and arc soil movement profile: a) bending moment profile; and b) $M_{\text{max}}$ against $w_s$. 116
Figure 4.29 Responses of 32 mm and 50 mm diameter piles subjected to arc soil movement profile ........................................................................... 117
Figure 4.30 Responses of 32 mm diameter pile subjected to different axial loads and arc soil movement profile.......................... 118
Figure 4.31 Responses of 50 mm diameter pile subjected to different axial loads and arc soil movement profile (0 N and 289 N) ......................... 119
Figure 4.32 Comparison of $p_{\text{max}}$ against limiting pressure profiles for laterally loaded piles .................................................................................................... 121
Figure 4.33 Responses of 50 mm diameter pile subjected to three soil movement profiles ($L_m = 400$ mm, $L_s = 300$ mm): d) $M_{\text{max}}$ against $w_s$; and e) $M_{\text{max}}$ against $S_{\text{max}}$. 125
Figure 4.34 Responses of 32 mm diameter pile subjected to three soil movement profiles ($L_m = 200$ mm, $L_s = 500$ mm): d) $M_{\text{max}}$ against $w_s$; and e) $M_{\text{max}}$ against $S_{\text{max}}$. 128
Figure 4.35 $M_{\text{max}}$ against $S_{\text{max}}$ relationship for single piles tests (rectangular, triangular and arc soil movement profiles)................................. 129
Figure 5.1 Flowchart showing the presentation of the pile group tests...................... 135
Figure 5.2 Test identification number for individual pile within a pile group ............ 136
Figure 5.3 Location of piles in a group and strain gauges position ............................ 139
Figure 5.4 Pile caps details ...................................................................................... 140
Figure 5.5 Location of a pile group in the shear box ................................................. 142
Figure 5.6 Response of 32 mm diameter piles in a row ($L_s/L_m = 200/500$) ............ 145
Figure 5.7 Bending failure on the connection between the hydraulic jack and the rectangular loading block.............................................................. 146
Figure 5.8 Response of 32 mm diameter piles in a line ($L_s/L_m = 400/300$) ............. 149
Figure 5.9 Response of 32 mm diameter piles in a line ($L_s/L_m = 200/500$) ......... 151
Figure 5.10 Response of 32 mm diameter piles in a line ($L_s/L_m = 400/300$) ............ 154
Figure 5.11 Response of $2 \times 2$ pile group at 3d pile spacing ($L_s/L_m = 200/500$) .... 157
Figure 5.12 Response of $2 \times 2$ pile group at 5d pile spacing ($L_s/L_m = 200/500$) .... 159
Figure 5.13 Response of $2 \times 2$ pile group at 3d pile spacing ($L_s/L_m = 400/300$) ..... 161
Figure 5.14 Response of $2 \times 2$ pile group at 5d pile spacing ($L_s/L_m = 400/300$) .... 163
Figure 5.15 Response of $2 \times 2$ pile group at 3d pile spacing ($L_s/L_m = 200/500$) .... 165
Figure 5.16 Response of $2 \times 2$ pile group at 5d pile spacing ($L_s/L_m = 200/500$) .... 167
Figure 5.17 The effect of $L_m/L_s$ ratio on the pile response of $1 \times 2$ pile group....... 171
Figure 5.18 The effect of $L_m/L_s$ on the bending moment profile of $2 \times 1$ pile group. 172
Figure 5.19 The effect of $L_m/L_s$ on the bending moment profile of $2 \times 2$ pile group 174
Figure 5.20 The effect of pile diameter on the bending moment profile of $2 \times 2$ pile group .............................................................................................. 176
Figure 5.21 The effect of pile spacing on the response of $1 \times 2$ pile group ............ 178
Figure 5.22 The effect of pile spacing on the response of 2 × 2 pile group............. 180
Figure 5.23 Forces and moments in piles A and B when subjected to lateral
soil movement.............................................................................................................. 181
Figure 5.24 The effect of pile spacing on the bending moment profile of
2 × 2 pile group ............................................................................................................. 184
Figure 5.25 Soil reaction at different w_s indicating load transfer between piles
A & B ............................................................................................................................... 185
Figure 5.26 The effect of pile spacing on the bending moment profile of
2 × 2 pile group ............................................................................................................. 188
Figure 5.27 The effect of pile spacing on the pile response in term of M_max
and r_max ......................................................................................................................... 189
Figure 5.28 The effect of pile group arrangement on the pile response .............. 193
Figure 5.29 Typical soil reaction profiles showing the r_max in L_m and L_s ............ 195
Figure 5.30 Maximum soil pressure in L_m for tests with L_m/L_s = 200/500 ............ 197
Figure 5.31 Maximum soil pressure in L_m for tests with L_m/L_s = 400/300 ............ 198
Figure 5.32 Maximum soil pressure in L_s for tests with L_m/L_s = 200/500 ............ 199
Figure 5.33 Maximum soil pressure in L_s for tests with L_m/L_s = 400/300 ............ 200
Figure 5.34 M_max against S_max relationship on the pile group tests ..................... 202
Figure 5.35 Total lateral force vs. frame movement...................................................... 204
Figure 5.36 Driving resistance during installation......................................................... 205
Figure 6.1 Mesh of soil and pile used in FLAC3D analysis ......................................... 211
Figure 6.2 Cantilever model pile calculated with static equations and FLAC3D ...... 214
Figure 6.3 The effect of mesh density on the pile response ......................................... 217
Figure 6.4 Comparison between Mohr-Coulomb and Drucker-Prager
constitutive models ....................................................................................................... 218
Figure 6.5 The effect of Young’s modulus on the pile response ............................... 220
Figure 6.6 The effect of dilation angle of sand on the pile response ......................... 221
Figure 6.7 The effect of interface elements on the pile response: (a)-(b) with
different interface strengths; and (c)-(d) with different interface stiffness...223
Figure 6.8 Pile responses of Test F_4-3_50_0k_600...................................................... 228
Figure 6.9 Contour showing the magnitude of soil and pile displacement ............... 229
Figure 6.10 The magnitude of stress (x-direction) build up in the shear box .......... 230
Figure 6.11 Pile responses of Test F_4-3_50_30k_600.................................................. 232
Figure 6.12 Pile responses of Test F_4-3_50_60k_600.................................................. 235
Figure 6.13 Force distribution from weight blocks to the pile head before and after the application of soil movement ............................................................. 236
Figure 6.14 Pile responses of Test F_2-5_32_0k_600...................................................... 238
Figure 6.15 Degree of sand dilating at different thickness of the moving
layer (L_m) ...................................................................................................................... 239
Figure 6.16 Pile responses of Test F_2-5_50_0k_600...................................................... 242
Figure 6.17 Contour showing the magnitude of soil and pile displacement
(\psi = 8^\circ) .................................................................................................................. 243
Figure 6.18 The magnitude of stress (x-direction) build up in the shear box
(\psi = 8^\circ) .................................................................................................................. 244
Figure 6.19 Response of the 50 mm diameter pile with different L_m/L_s ratio ....... 246
Figure 6.20 Comparison of M_max at Q = 0 N between FLAC3D and Experimental
results: (a) w_s = 30 mm; (b) w_s = 60 mm; (c) w_s = 90 mm; (d) w_s = 120 mm;
and (e) Summary of FLAC3D analysis ....................................................................... 249
Figure 6.21 Comparison of maximum pile deflection at Q = 0 N between
FLAC3D and Experimental results: (a) w_s = 30 mm; (b) w_s = 60 mm;
(c) \( w_s = 90 \text{ mm} \); (d) \( w_s = 120 \text{ mm} \); and (e) Summary of FLAC\textsuperscript{3D} analysis.

Figure 6.22 Comparison of maximum shear force at \( Q = 0 \text{ N} \) between FLAC\textsuperscript{3D} and Experimental results: (a) \( w_s = 30 \text{ mm} \); (b) \( w_s = 60 \text{ mm} \); (c) \( w_s = 90 \text{ mm} \); (d) \( w_s = 120 \text{ mm} \); and (e) Summary of FLAC\textsuperscript{3D} analysis.

Figure 6.23 Comparison of maximum soil reaction at \( Q = 0 \text{ N} \) between FLAC\textsuperscript{3D} and Experimental results: (a) \( w_s = 30 \text{ mm} \); (b) \( w_s = 60 \text{ mm} \); (c) \( w_s = 90 \text{ mm} \); (d) \( w_s = 120 \text{ mm} \); and (e) Summary of FLAC\textsuperscript{3D} analysis.

Figure 6.24 Comparison of \( M_{\text{max}} \) at \( Q = 294 \text{ N} \) between FLAC\textsuperscript{3D} and Experimental results: (a) \( w_s = 30 \text{ mm} \); (b) \( w_s = 60 \text{ mm} \); (c) \( w_s = 90 \text{ mm} \); (d) \( w_s = 120 \text{ mm} \); and (e) Summary of FLAC\textsuperscript{3D} analysis.

Figure 6.25 Comparison of maximum pile deflection at \( Q = 294 \text{ N} \) between FLAC\textsuperscript{3D} and Experimental results: (a) \( w_s = 30 \text{ mm} \); (b) \( w_s = 60 \text{ mm} \); (c) \( w_s = 90 \text{ mm} \); (d) \( w_s = 120 \text{ mm} \); and (e) Summary of FLAC\textsuperscript{3D} analysis.

Figure 6.26 Comparison of maximum shear force at \( Q = 294 \text{ N} \) between FLAC\textsuperscript{3D} and Experimental results: (a) \( w_s = 30 \text{ mm} \); (b) \( w_s = 60 \text{ mm} \); (c) \( w_s = 90 \text{ mm} \); (d) \( w_s = 120 \text{ mm} \); and (e) Summary of FLAC\textsuperscript{3D} analysis.

Figure 6.27 Comparison of maximum soil reaction at \( Q = 294 \text{ N} \) between FLAC\textsuperscript{3D} and Experimental results: (a) \( w_s = 30 \text{ mm} \); (b) \( w_s = 60 \text{ mm} \); (c) \( w_s = 90 \text{ mm} \); (d) \( w_s = 120 \text{ mm} \); and (e) Summary of FLAC\textsuperscript{3D} analysis.

Figure 6.28 The effect of sand dilation on the \( M_{\text{max}} \) at different \( L_m/L_s \) ratios: (a) \( w_s = 20 \text{ mm} \); (b) \( w_s = 60 \text{ mm} \); (c) \( w_s = 90 \text{ mm} \); and (d) \( w_s = 120 \text{ mm} \); and (e) Summary of FLAC\textsuperscript{3D} analysis with \( \phi = 38^\circ \).

Figure 6.29 \( M_{\text{max}} \) against \( S_{\text{max}} \) with increasing \( L_m/L_s \) (\( w_s = 60~120 \text{ mm} \)).

Figure 6.30 \( M_{\text{max}} \) against \( S_{\text{max}} \) for two sets of Flac\textsuperscript{3D} models: 1) \( \psi = \phi \); and 2) \( \psi = \phi - 30^\circ \) (\( w_s = 60 \text{ mm} \), \( L_m/L_s = 100/600~500/200 \)).

Figure 7.1 Dimensions of shear box used for pile testing in clay.

Figure 7.2 Test identification number for single piles and pile groups in clay.

Figure 7.3 Locations of vane shear tests in the shear box.

Figure 7.4 Average shear strength of clay in the shear box.

Figure 7.5 Pile response of Test C_2-5_25_170805.

Figure 7.6 Pile response of Test C_2-5_25_250605.

Figure 7.7 Pile response of Test C_4-3_32_290705.

Figure 7.8 \( M_{\text{max}}/c_u d^3 \) against pile diameter (\( L_m/L_s = 200/500 \)).

Figure 7.9 Pile response of Test C_2-5_32_1×2_3d_A_230705 (pile A).

Figure 7.10 Pile response of Test C_2-5_32_1×2_3d_B_230705 (pile B).

Figure 7.11 Pile response of Test C_2-5_32_1×2_5d_A_300705 (pile A).

Figure 7.12 Pile response of Test C_2-5_32_1×2_5d_B_300705 (pile B).

Figure 7.13 Pile response of Test C_2-5_32_2×1_3d_A_270705 (pile A).

Figure 7.14 Pile response of Test C_2-5_32_2×1_3d_B_270705 (pile B).

Figure 7.15 Comparison of normalised bending moment profiles between pile groups.

Figure 7.16 Comparison of normalised soil reaction profiles between pile groups.

Figure 7.17 Soil plug into the pile tip.

Figure 8.1 Limiting pressure profiles for sand.

Figure 8.2. Piles and pipelines subjected to soil movement.

Figure 8.3. Different failure modes of rigid pile.

Figure 8.4. Experimental and numerical evidence of the failure modes on piles subjected to lateral soil movement.

Figure 8.5 Pressure distribution profiles in the moving layer (\( L_m \)).
Figure 8.6 Pressure profiles in the active section of the pile by Prasad and Chari (1999) .............................................................303
Figure 8.7 Flowchart for using the current simplified solution ..................................................305
Figure 8.8 Response of 32 mm diameter pile subject to rectangular profile soil movement (L_m/L_s = 200/500) .................................................................307
Figure 8.9 Response of 50 mm diameter pile subject to rectangular profile soil movement (L_m/L_s = 200/500) .................................................................309
Figure 8.10 Case 1: The predicted and the measured pile responses for Chen (1994) .................................................................312
Figure 8.11 Case 2: The predicted and the measured pile responses for Chen (1994) .................................................................313
Figure 8.12 Case 3: The predicted and the measured pile responses for Tsuchiya et al (2001) .................................................................316
Figure 8.13 Plan view of the sliding zone and location of the instrumented piles and inclinometers (after Kalteziotis et al, 1993) ..................317
Figure 8.14 Lateral soil movement profile (after Kalteziotis et al, 1993) ..................318
Figure 8.15 Case 4: The predicted and the measured pile responses for Kaltezoitis et al (1993) .................................................................319
## LIST OF TABLES

Table 2.1 Summary of two-dimensional numerical studies on piles subjected to lateral soil movements, 1981 to 2001 .......................................................16
Table 2.2 Summary of three-dimensional numerical studies on piles subjected to lateral soil movements, 1983 to 2005.............................................17
Table 2.3 Summary of the reported laboratory model scale and full-scale tests on piles subjected to lateral soil movements, 1974 to 1990 .....29
Table 2.4 Studies on axially loaded piles subjected to either lateral load or lateral soil movement.............................................................................33
Table 3.1 Magnitude of soil movement and the corresponding depth of sliding layer for the triangular soil movement profile .......................41
Table 3.2 Magnitude of soil movement and the corresponding depth of sliding layer for the arc soil movement profile ..........................45
Table 3.3 Structural properties of the instrumented model piles .................................................................61
Table 4.1 Tests conducted on single piles with rectangular soil movement profile ..........................79
Table 4.2 Tests conducted to study the boundary effect of the shear box ..................................................................................81
Table 4.3 Tests conducted on single piles with the triangular (T) soil movement profile ............................101
Table 4.4 Tests conducted on single piles with the arc (P) soil movement profile ................................................................................110
Table 5.1 Tests conducted on piles in a group ..................................................................................135
Table 5.2 Reference to the 2 × 2 pile groups tests ........................................................................168
Table 5.3 Summary of the effect of Lm/Ls ratio on the piles in a row (w_s = 60 mm) 171
Table 5.4 Summary of the effect of Lm/Ls ratio on the piles in a line (w_s = 60 mm) 172
Table 5.5 Summary of the effect of Lm/Ls ratio on the piles in a square (w_s = 60 mm) ...................................................................................................174
Table 5.6 Summary of the effect of pile diameter on the piles (w_s = 60 mm)...........................................176
Table 5.7 Summary of the effect of the pile spacing on the piles (w_s = 60 mm)...................................182
Table 5.8 Summary of the effect pile spacing on the piles (w_s = 60 mm)...........................................184
Table 5.9 Summary of load transfer between piles A and B ..................................................................................186
Table 5.10 Summary of the effect of the pile spacing on the piles (w_s = 60 mm)...................................188
Table 5.11 Comparison between different pile group arrangements with single pile (w_s = 60 mm) ..................................................................................194
Table 5.12 General pile behaviour trends identified in the pile group tests ..................................................208
Table 5.13 General pile behaviour trends in the pile group tests (continue) ..209
Table 6.1 Input parameters for the standard model ..............................................................................215
Table 6.2 Different sand properties used in parametric study ........................................................................219
Table 6.3 Summary of the results obtained from the parametric study ..........................................................224
Table 6.4 FLAC3D analyses performed on model tests ........................................................................225
Table 6.5 FLAC3D models with Q = 0 N at different ratio of Lm/Ls ..........................................................245
Table 6.6 M_max = αS_max relationship at different Lm/Ls ..................................................................262
Table 7.1 Properties of kaolin clay .................................................................................272
Table 7.2 Tests on single piles and pile groups in clay subjected to uniform soil movement profile ..............................................................................275
Table 7.3 Summary of single pile test results (w_s = 60 mm) .................................................................282
Table 7.4 Response of piles A and B at w_s = 130 mm ....................................................................283
Table 7.5 Summary of the response of piles A and B at w_s =130 mm .......................................................286
Table 7.6 Summary of the response of piles A and B at w_s = 130 mm .......................................................287
Table 8.1. Soil movement to mobilise limiting pressure on pipelines........................................299
Table 8.2 Summary of pile and soil properties used in the simplified solution........306
Table 8.3 Summary of the critical pile responses with different soil
movement profiles ................................................................................................307
Table 8.4 Comparison between the predicted results from current solution
and the measured results from the model test.....................................................308
Table 8.5 Input parameters and comparison between results for Case 1...............311
Table 8.6 Input parameters and comparison between results for Case 2.................313
Table 8.7 Input parameters and results for Case 3..............................................315
Table 8.8 Input parameters and results for Case 4..............................................320
ABSTRACT

Many studies have been undertaken on piles subjected to vertical load, and on piles subjected to lateral soil movement. However, little information is available for evaluating the response of vertically loaded piles due to soil movement. In this thesis studies have been carried out to investigate the behaviour of axially loaded piles subjected to lateral soil movements.

The main focus of the thesis was experimental, where a testing apparatus had been fabricated to perform laboratory model tests in either sand or clay. In each test, the apparatus had the ability to vary a number of parameters such as: the shape of the soil movement profile; the axial load; the ratio of the moving to the stable soil layers; the pile diameter; and the soil properties. In addition to these parameters, the pile group tests involved different pile spacing and various pile group arrangement. The pile response, in terms of bending strain and displacement at the pile head, was measured.

The vast majority of tests were conducted on piles driven into sand. The results from the single pile tests show that, regardless of the shape of the soil movement profile, the maximum bending moment increases with the pile diameter, the sand density and the axial load. The ratio of the moving to the stable soil layers changed with the magnitude and the shape of the bending moment profiles. The pile group tests, apart from the abovementioned parameters, were found to have an influence on the piles; the pile spacing and the pile group arrangement also influenced the behaviour of the individual piles in a group. The testing apparatus was modified to conduct the tests in unconsolidated clay. The clay was very soft, which resulted a high relative stiffness between the pile and the clay. The results from a small number of tests showed that, in single pile tests, the piles behave like a rigid pile with the maximum bending moment increasing with the pile diameter.

Numerical analyses with the three-dimensional finite difference method were performed to predict and compare the results from the single pile tests in sand. In parametric studies it was found that Young’s modulus, friction angle, dilation angle were among the parameters that had the greater effect on pile behaviour, in particular
the maximum bending moment. Other less influential parameters were the interface properties between the pile and the soil, the density of the brick elements, and constitutive models. In all the predictions, the numerical analysis was shown to be able to predict the experimental results reasonably well. A further investigation into soil parameters (yet to be assessed in the experimental studies) was also carried out. These parameters included the ratio of the moving soil to the stable soil layers and the dilation angle of the sand.

A simplified solution was proposed to analyse a rigid pile subjected to lateral soil movement by the mean of calculating the pressure distribution acting along the pile shaft. The simplified solution was able to predict the bending moment and shear force profiles from the calculated pressure distribution acting along the pile. A number of case studies were carried out to test the ability of the simplified solution.

The experimental and numerical studies undertaken for this thesis have provided an important understanding of the behaviour of axially loaded piles subjected to lateral soil movement. In hope to benefit future research work, the limitations of the current studies and the areas for further research are outlined.
ACKNOWLEDGEMENTS

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Declaration

This work has not previously been submitted for a degree or diploma in any university. To the best of my knowledge and belief, the thesis contains no material previously published or written by another person except where due reference is made in the thesis itself.

_____________________
Eng How Ghee
14th October 2009
LIST OF PUBLICATIONS

Below is the list of nine papers published during the course of my study:


NOTATIONS

- **B**  breadth or width of the pile
- **bd** boundary distance from the inner face of the shear box wall (adjacent to the lateral hydraulic jack)
- **Cc** coefficient of curvature of sand particles
- **Cu** coefficient of uniformity of sand particles
- **ci** cohesion (in Coulomb’s criterion) of interface element in FLAC3D
- **cu, su** undrained shear strength
- **CL** centreline
- **d** pile diameter
- **D** depth of the pile
- **D1** centre-to-centre spacing between two piles
- **D2** edge to edge spacing between two piles
- **D_{cons}** constrained modulus
- **Dr** relative density of sand
- **EI (EpIp)** pile bending stiffness
- **Ec** Young’s modulus of concrete pile
- **Ep** Young’s modulus of pile
- **Es** Young’s modulus of soil
- **Es_{Lm}** Young’s modulus of soil in moving layer
- **Es_{Ls}** Young’s modulus of soil in stable layer
- **Fs** factor of safety against failure
- **F^m** group factor in terms of maximum positive bending moment
- **F’** group factor in terms of maximum positive bending moment
- **g** gravity (9.81 m/sec^2)
G, Gs  shear modulus of soil
H     height of embankment or retaining wall
H'    actual embankment height
I_p   second moment of area of pile
K, Ks  bulk modulus of soil or soil parameter
k     modulus of lateral soil reaction or modulus of subgrade reaction
K_o   coefficient of earth pressure at rest
K_p   coefficient of passive earth pressure
k_n   normal stiffness
K_R   pile flexibility factor (= E_p I_p / E_s L^4)
k_s   shear stiffness
L     pile length = L_m + L_s
L_m   pile embedded length in upper moving soil layer
L_s   pile embedded length in upper stable soil layer
M_{max}  maximum bending moment (absolute value of either M_{max} or M_{max})
M_{max,i} maximum bending moment for a pile in a group
M_{max,s} maximum bending moment of the “standard” single pile
M_{max}  maximum negative bending moment
M_{+max} maximum positive bending moment
N_h   rate of increase of Young’s modulus with depth (E_s / z)
N_p   lateral capacity factor
P     lateral loading applied on the pile head
r_d   Coefficient of determination
r     lateral soil reaction (or lateral soil pressure × pile diameter, p.d)
r_{max} maximum lateral soil reaction (absolute value of either r_{max} or r_{+max})
\( r_{\text{max } i} \) maximum lateral soil reaction of a pile ‘i’ in a group

\( r_{\text{max } s} \) maximum lateral soil reaction of the single pile

\( r_{-\text{max}} \) maximum negative lateral soil reaction acting on pile

\( r_{+\text{max}} \) maximum positive lateral soil reaction acting on pile

\( r_{\text{ult}} \) ultimate lateral soil reaction (or ultimate lateral soil pressure \( \times \) pile diameter)

\( r_d^2 \) coefficient of uniformity of a linear fit equation

\( p \) or \( p_{h} \) lateral soil pressure (lateral soil reaction/pile diameter, \( r/d \))

\( p_{h(\text{max})} \) maximum lateral soil pressure (lateral soil reaction/pile diameter, \( r/d \))

\( p_{p} \) Rankine passive earth pressure (\( = K_p \gamma H \))

\( p_{\text{max } i} \) maximum lateral soil pressure of a pile ‘i’ in a group

\( p_{\text{max } s} \) maximum lateral soil pressure of the single pile

\( p_{-\text{max}} \) maximum negative lateral soil pressure acting on pile

\( p_{+\text{max}} \) maximum positive lateral soil pressure acting on pile

\( p_{\text{max}} \) maximum absolute value of lateral soil pressure (\( p_{\text{max}} = r_{\text{max}}/d \))

\( p_{u} \) ultimate lateral soil pressure

\( Q \) applied axial load on the pile head (single pile) or top of pile cap (pile group)

\( S.D. \) sliding depth interface between the moving and the stable layers

\( S \) shear force

\( S_{\text{max}} \) maximum shear force (absolute value of either \( S_{+\text{max}} \) or \( S_{-\text{max}} \))

\( S_{+\text{max}} \) maximum positive shear force

\( S_{-\text{max}} \) maximum negative shear force

\( s_u \) undrained shear strength

\( t_i \) tensile strength at interface

\( w_s \) lateral soil movement (unit in mm)
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$y$ or $y_p$</td>
<td>lateral or horizontal pile displacement</td>
</tr>
<tr>
<td>$y_m$</td>
<td>lateral displacement at point m</td>
</tr>
<tr>
<td>$y_s$</td>
<td>lateral displacement of soil</td>
</tr>
<tr>
<td>$z$</td>
<td>depth below soil/ground surface</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>coefficient of linear fit for $M_{\text{max}} = \alpha S_{\text{max}}$ relationship</td>
</tr>
<tr>
<td>$\beta$</td>
<td>embankment front slope or a non-dimensional quantity</td>
</tr>
<tr>
<td>$\phi_i$</td>
<td>angle of internal friction of interface element in FLAC$^{3D}$</td>
</tr>
<tr>
<td>$\phi, \phi_i$</td>
<td>angle of internal friction of sand (peak)</td>
</tr>
<tr>
<td>$\psi$</td>
<td>angle of dilation of sand</td>
</tr>
<tr>
<td>$\psi_i$</td>
<td>angle of dilation of interface element in FLAC$^{3D}$</td>
</tr>
<tr>
<td>$\gamma'$</td>
<td>actual embankment density or average effective unit weight from ground surface</td>
</tr>
<tr>
<td>$\gamma, \gamma_s$</td>
<td>soil unit weight of soil (kN/m$^3$)</td>
</tr>
<tr>
<td>$\gamma_p$</td>
<td>unit weight of the pile (kN/m$^3$)</td>
</tr>
<tr>
<td>$\nu, \nu_s, \nu_p$</td>
<td>Poisson’s ratio, soil and pile</td>
</tr>
<tr>
<td>$\sigma_h$</td>
<td>horizontal normal stress</td>
</tr>
<tr>
<td>$\sigma_t$</td>
<td>tension limits or tensile strength</td>
</tr>
<tr>
<td>?</td>
<td>possible error in reading or data</td>
</tr>
<tr>
<td>-ve</td>
<td>negative magnitude or value</td>
</tr>
<tr>
<td>+ve</td>
<td>positive magnitude or value</td>
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</table>
Chapter 1: Introduction

1.0 Introduction

Pile foundations may be designed to provide resistance against lateral soil movements. These piles are commonly known as passive piles and can be found in deep excavation activities, tunneling operations, pile driving operations, moving slopes, settling embankments, offshore foundations and liquefiable soils. In 1977, the Committee of International Society of Soil Mechanics and Foundation Engineering, during the 9th international conference in Tokyo, organised a Specialty Session (De Beer et al, 1977) as a way to share knowledge on passive piles. The Specialty Session focused on two major areas: the effects of adjacent surcharge loadings on the piles; and the effects of horizontal dynamic loading, and earthquakes, on piles. More than 30 years later, there has been a considerable amount of research undertaken in this field.

Importantly, pile foundations, designed to support tall buildings and services, are often subjected to lateral soil movements and axial loads, simultaneously. While many studies have addressed, specifically, piles subjected to vertical load, and subjected to lateral soil movement, little information is available in terms of evaluating the response of vertically loaded piles subjected to soil movements. For example, rows of either isolated or contiguous piles that are commonly used for basement walls; the pressure of the retained soil and the weight from the superstructures, are imposed simultaneously to the piles. This problem of combined loadings on the piles has considerable limited amount of research work been undertaken. To date, studies on piles related to the liquefaction induced lateral soil movement have found that the axial load on the piles can cause: (1) additional bending moments; (2) additional compression stresses; and (3) additional lateral displacements (Bhattacharya, 2003). Other studies have found that lateral soil movements may increase the axial capacity of piles (Chen et al, 2002). Such an increase is similar to that noted for a laterally loaded pile, subjected to an axial load. Experimental testing has been undertaken by Anagnostopoulos and Georgiadis (1993), while numerical simulations have been undertaken by Trochanis et al (1991).
Chapter 1: Introduction

Generally, there are many readily available solutions (empirical, semi-empirical and analytical) for the analysis and the design of the piles. However, the current design assumptions treat the combination of the loads, acting on the piles, individually. For an example, for the contiguous piles used in basement walls, in respect to the amount of pressure from the retaining soil acting on the piles, the ultimate axial resistance of the pile is unaffected.

In order to address the combined effects from the loads acting on the piles, an analysis can be achieved through the use of sophisticated numerical approaches, such as finite element or finite difference methods (e.g. ADINA, ABAQUS, CRISP, FLAC and PLAXIS). In the case of a two-dimensional analysis, these might only approximately account for what are three-dimensional interactions. With the recent development of three-dimensional packages (e.g. ABAQUS, FLAC$^{3D}$ and PLAXIS$^{3D}$), a complete solution is now achievable. Although these approaches are accepted in both academia and engineering practice, they are found to be time-consuming and costly, and are warranted only for complex and critical geotechnical problems. The alternative boundary element approach (Poulos and Davis, 1980) is based on the availability of relevant closed form solutions (e.g. Mindlin, 1936; Chan et al, 1974). Although this is a very efficient technique, rigorous numerical solutions are limited to relatively simplified cases, thus precluding its application to most real projects where complex foundation conditions exist. The development of this numerical approach is, therefore, dependent on the development of a wider array of closed form solutions (Guo, 1997). Additionally, the success of these numerical approaches is fundamentally dependent on the accuracy of the constitutive models adopted. In many cases, the parameters required for these models cannot be easily determined, and must be estimated or guessed.

Previous experimental investigations, although focused on a very specific problem (e.g. Steward, 1992; Chen, 1994; Leung et al, 2000), nevertheless, provided useful data to calibrate the numerical analysis. The current design approach without considering the interaction between the axial load and the soil movement may be uneconomical, and/or unsafe. To provide insight into the mechanism of this interaction, and to gain a greater understanding of the behaviour of the pile under the combined loadings, comprehensive experimental and numerical research is needed.
1.1 Aim and objectives

The aim of the current research project was to study the behaviour of axially loaded piles subjected to lateral soil movements. To understand the pile behaviour the parameters contributing to the pile response, in terms of bending moment, shear force, soil reaction, rotation and pile deflection, first needed to be identified.

The major objectives of this project were:

**Experimental program**

The current research program consisted of experimental work, with the focus being to design and establish a new testing apparatus, as well as to subsequently conducting and interpreting model tests on single and pile groups embedded in either sand or clay. The test data, obtained from the experiment were used to identify a number of parameters known to have an effect on the pile or pile group behaviours.

**Numerical program**

The numerical work associated with this research project focused on the back analysis of the model tests. A comprehensive series of analyses, using the finite difference method, were undertaken to examine the suitability of the technique for modeling the problem. Subsequent studies with the numerical program addressed the effect of the moving to the stable layers ratio and the dilation angle of sand on the pile behaviour.

**Analytical program**

Knowing the important parameters had contributed to the pile behaviour, the analytical work focused on developing a simplified solution to analyse the single piles subjected to lateral soil movement. The newly developed simplified solution was initially used for the back analysis of the experimental model tests. Subsequently, the simplified solution was used to perform case studies on the reported model and the full-scale tests.
1.2 Thesis outline

The thesis outline, consisting of nine chapters, described below.

Chapter 1: *Introduction* provides a brief introduction of the existing works and outlines the main aim and objective of the current research.

Chapter 2: *Literature review* presents a critical review of the published literatures, categorised into three areas: theoretical, numerical and experimental. This review of existing published experimental work provides useful ideas for the design of the new apparatus design, while the review of the numerical work provides useful information on the current limitations and advantages of different numerical models and packages being used.

Chapter 3: *Experimental setup* describes the design of the new experimental apparatus, and the standard procedures used to undertake the model tests in sand. The chapter focuses mainly on single pile tests, while the different procedures and modifications to the testing apparatus for pile group tests and tests in clay are described in their respective chapters. A number of calibrations on the model pile and the sand properties obtained from the standard laboratory tests are also described. Finally, the methods used for the data interpretation, to obtain the pile responses, are described.

Chapter 4: *Single pile tests* presents and discusses the results of a series of model tests undertaken on single piles embedded in sand.

Chapter 5: *Pile group tests* describes a number of extra procedures used for undertaking the pile group tests and, subsequently, presents and discusses the results of a series of pile group tests in sand.

Chapter 6: *Three-dimensional finite difference analysis* describes the three-dimensional numerical package used to predict the single piles tests in sand. The results obtained from the numerical analysis are then compared to the single pile tests,
with a discussion on the findings being presented. Additional investigations into the other unexplored parameters, that affect the pile behaviour, are also described.

Chapter 7: *Single pile and pile group tests in clay* describes the modifications to the new experimental apparatus in sand, that are required for its use in conducting single pile and pile group tests in clay. Subsequently, the results and discussions on the tests are presented.

Chapter 8: *Simplified solution to analysis rigid pile subjected to lateral soil movement* describes the formulation of a simplified solution for the analysis of single piles subjected to lateral soil movement. A number of case studies, undertaken to predict the results from the published model and full-scaled tests, are outlined.

Chapter 9: *Conclusions and recommendations* presents the major findings from this research, and suggestions on the areas for improvement and additional work that may be beneficial for future research are highlighted.

The standard laboratory tests performed to investigate the properties of the sand used in the model tests are presented in Appendix A. The test results for the single pile and pile group tests are compiled in Appendixes B and C, respectively. In Appendix D, the input data file for the standard model used in the FLAC\textsuperscript{3D} is shown.
Chapter 2: Literature Review

2.0 Introduction

Since the birth of geomechanics, considerable research has been devoted to unstabilised slope analysis; however, recently efforts have shifted to analysis of stabilised slopes (e.g. Briaud and Lim, 1997; Liang, et al., 1998). In many cases, beams (piles, anchors, or soil nails) are often needed to stabilise a slope (Ito and Matsui, 1975; Chen and Poulos, 1997). Unfortunately, useful design expressions or guidelines for such beams are scarce, and estimations of the resistance force of a beam for a given amount of sliding soil are particularly difficult. As noted previously, contradictory principles have also been practiced; especially in the designing of a piled slope, a soil nailed slope, and a pipeline within unstable soil. Therefore, the design for a pile slope or a pipeline may be inefficient, uneconomical, and even irrational. Thus, a unified approach to soil nail, pipeline and lateral pile due to soil movement is needed to eliminate the existing contradictory principles, with the outcome being more efficient, and economic design.

2.1 Analysis methods

The analysis methods currently available to analyse piles subjected to lateral soil movements can be categorised into the following methods:

1) pressure based method
2) displacement based method
3) finite element method.

2.1.1 Pressure based methods

A range of numerical approaches have been developed to analyse the response of piles due to soil movements in diverse situations, e.g. for stabilising a slope (Poulos, 1995), for retaining an excavation (Chen and Poulos, 1997), and for offshore structure foundations (Bryne et al., 1984). The approach depends largely on the (limiting) pressure assumed for that part of the pile in the moving layer. This limiting pressure has also been estimated by only the few available simple methods, a number of which
are largely empirical, with most allowing only the maximum pile bending moment to be estimated. The following section outlines six (a - f) of these simple methods:

\[ a) \]  \textit{Begemann and De Leeuw's method}

Begemann and De Leeuw (1972) suggested a relatively simple method for estimating the pressure distribution applied to piles in relation to the surface surcharge, taking into consideration the stiffness of the piles. The lateral soil displacement and horizontal stress profiles are calculated by an elastic method, ignoring the presence of the pile (Figure 2.1). An average value of horizontal stress and the maximum lateral displacement are used to define the line A-B (Figure 2.1), where the pile is considered as an equivalent sheet pile wall. Point A corresponds to a rigid wall with no displacement and, therefore, a horizontal stress of twice the elastic value would be applied to the wall. Point B corresponds to a very flexible wall with the displacement equal to the free soil and, therefore, no horizontal force would be acting on it. The line O to C describes the deformability of the pile under the action of a uniformly distributed load, with simple assumptions regarding the end fixity conditions.

The point of intersection of the two lines gives the solution for compatibility between the pile and the soil displacements, which form the maximum pile bending moment, can be calculated. This compatibility is strictly only applicable at the point of maximum displacement and is only approximate as it does not allow for relative movement between the pile and the surrounding soil, nor does it allow a true representation of the pressure distribution acting on the pile. More complex methods that allow for compatibility between pile and soil displacements along the length of the piles are described in a later section of this report.
b) **De Beer and Wallays’ method**

De Beer and Wallays’ (1972) empirical design method took into account the geometry of the embankment and the distance of the piles from the toe (as shown in Figure 2.2). The method assumed that a uniform pressure acts on the piles over the full depth of the soft layer. This pressure was used in conjunction with very simple assumptions regarding pile head fixity, as well as support from the stronger soil layer to estimate the maximum pile bending moment. However, the variation of the bending moment along the pile could not be calculated. A comparison of this simple design method with some field studies showed relatively good predictions of the maximum bending moment when the factor of safety against failure of the embankment is greater than 1.6. Thus De Beer and Wallays (1972) developed a second method for cases when the factor of safety is less than 1.6. In such a case the most critical slip circle is drawn and the full ultimate soil pressure \((10.5s_u)\) is assumed to act on the pile above the point where the slip circle crosses the pile (as shown in Figure 2.3).
Tschebotarioff’s (1973) design recommendations were similar to the first De Beer and Wallyas’ (1972) method. Originally the method recommended that a triangular shaped lateral soil pressure be calculated (as shown in Figure 2.4). A revision was in the light of subsequent field measurements. $\sigma_z$ was taken as the increase in vertical stress at the mid depth of the soft layer at the location of the piles; the suggested lateral soil pressure could be ignored when the embankment loading was less than $3s_u$ (corresponding roughly to a factor of safety of 1.7).
d) **Ito and Matsui’s method**

Ito and Matsui’s (1975) theoretical method analysed the growth mechanism of the lateral force acting on stabilising piles in a row, due to the surrounding plastic deformation. The pressure acting on the piles was estimated on the basis of the extrusion of soil between the neighbouring piles. As the piles were founded in a relatively deep layer of soft soil, they were assumed to follow the deformation of the soil. As shown in Figure 2.5, the piles of same diameter (d) were placed in a row with a centre-to-centre spacing $D_1$ through plastically-deforming ground ($D_2$ is the edge to edge spacing between the two piles). When a lateral deformation occurred within a soil layer of thickness $H$, in a direction perpendicular to that of the row of piles, the lateral forces are assumed to act on the piles as an interaction between the piles and soil layer. In such a case, it was sufficient to analyse the behaviour of a soil layer between two piles.
Viggiani’s method

Viggiani (1981) analysed the mechanism of interaction between a sliding soil mass and a pile crossing it, then penetrating the stable underlying soil, in which six possible failure modes were proposed. The piles, whose yield moment was greater than the bending moment acting upon them, were considered as rigid piles; and three possible soil failure modes were proposed. The occurrence of one of the six failure modes was perceived to be governed by the geometry of the problem (length and diameter of the pile, thickness of the sliding soil mass) on the yield moment of the pile section and on the undrained shear strength of the stable and sliding soil. The pressure distribution was assumed for each failure mode and, accordingly, shear force and bending moment in the piles were computed by the limit equilibrium method. The ultimate soil pressure ($p_u$), can be computed from the expression:

$$p_u = N_p s_u$$

Where $N_p$ is the lateral capacity factor, and $s_u$ is the undrained shear strength of the soil.

By examining the case of the lateral loads acting on the piles used to stabilize the landslides, Viggiani proposed that $p_u$ would vary, depending on whether the pile was
actively (6.26 - 12.56s_a) or passively (2.8 - 4.0s_a) loaded (from the studies of previous researchers).

f) **Springman’s method**

Springman’s (1989) simple design method resulted from centrifugal model tests on piles found adjacent to an embankment. The method predicts a parabolic shaped lateral soil pressure distribution action on the piles, taking into consideration the average differential soil-pile movement (as shown in Figure 2.6).

![Figure 2.6 Method of Springman (1989) (after Steward, 1992)](image)

**g) Steward’s method**

Steward (1992) later proposed a modification to Springman’s (1989) method by taking into account the pile deflection profile on pile with low relative soil-pile stiffness had on the pressure distribution in the soft soil layer.

Rather than applying a parabolic distribution pressure onto the entire length of the pile section, in the soft soil layer (see Figure 2.6, Springman, 1989), a uniform distribution pressure was applied, only on the pile section, where the soil displacement exceeded the pile deflection. Since the shape of the pressure distribution having little effect on the bending moment profile of the pile, Steward (1992) had not refined the uniform distribution pressure to that parabolic distribution pressure used in Springman (1989).
2.1.2 Displacement based methods

The second analysis method, the displacement based method, provides a more comprehensive analysis of the soil-pile interaction. Here, the free-field soil displacement profile, in the absence of the piles, must be estimated or measured. The displacement profile may be superimposed on the pile and the interaction calculated with either an elastic or subgrade reaction analysis. The advantage of this method is that it allows bending moment and deflection profiles to be calculated and, accordingly, the shear force and soil pressure distribution profiles.

a) Elastic analysis

Poulos’ (1973) finite difference boundary element analysis (BEM) assesses a single pile embedded in an elastic soil. The pile is represented as a thin strip interaction with the soil calculated from the equation, proposed by Mindlin (1936), for a semi-infinite elastic mass. A limiting pressure that can act on the pile is specified to account for the plastic yield around the pile. The method accounts for the continuity of the soil mass and allows the soil properties to vary with the depth to be incorporated into the calculation.

b) Subgrade reaction analysis

Marche’s (1973) subgrade reaction analysis (the more commonly used for the analysis of laterally-loaded pile) analyses piles subjected to lateral soil movements. This method is different from the elastic method (Poulos, 1973), in which soil continuity is not directly accounted for, although the selection of the appropriate subgrade moduli, approximately, allows for the interaction effects. The governing equation for this method is:

$$E_p I_p \frac{d^4 y_p}{dz^4} = k(y_p - y_s)$$

Where $k$ is the modulus of subgrade reaction, $y_p$ is the lateral displacement of the pile and $y_s$ is the lateral displacement of soil. A good agreement was found with the field data by using the measured free field soil displacements and by accounting for the variation in the soil stiffness with depth.
Maugeri and Motta’s (1992) extended Marche’s (1973) method by representing the linear subgrade reaction, k, with a nonlinear hyperbolic function, together with a limiting load of the soil-pile interaction. Byrne et al (1994) also proposed to represent k with non-linear p-y curves used for the analysis of a laterally-loaded pile.

Additionally, Cai and Ugai (2003) extended the subgrade reaction method for an analysis of the piles used to stabilise landslides. Their solution considered the influence of the linear soil movement profile of the moving layer.

2.1.3 Finite element methods

The third analysis method is represented by the finite element method in plane-strain analysis, with refinements developed over a number of years in terms of the improvement of computer hardware and software packages.

b) Plane-strain

Rowe and Poulos (1979) described a plain-strain finite element analysis for the analysis of the undrained behaviour of soil slopes reinforced by pile groups. In the analysis, the soil and the piles were considered separately and were related by nodal forces satisfying compatibility and equilibrium requirements.

Randolph (1981) performed a site-specific plane-strain analysis, where the piles were replaced by an equivalent sheet-pile wall, with a flexibility equal to the average of the piles and the soil it replaced. The analysis can be used to analyse piles in groups by incorporating them into the finite element mesh.

Finno et al (1991) used a plane-strain finite element program to study the behaviour of a tieback sheet-pile wall adjacent to a neighbouring deep excavation. The program modelled the excavation process to predict the soil and pile-cap movements. First the pile deformations were assessed, then the computed deformations were used to compute the bending moments in the piles to evaluate the effects of the movements on their integrity. The prediction and the field data agreed fairly well.
Meyersohn (1994) developed a finite element model to study the response of single piles and pile groups subjected to lateral ground movements associated with soil liquefaction. The capability of the model was compared with a number of published test data. The study identified three failure mechanisms under the lateral spread condition, which are a function of the relative stiffness between the soil and the pile, and the axial load imposed on the pile and thickness of the liquefied soil. One of the failure mechanisms identified was the buckling that developed as a result of the high axial load and lateral pile displacement induced by the soil movement.

Due to space limitation, the current published two-dimensional and three-dimensional numerical studies identified in the literature review are summarised in Tables 2.1 and 2.2, respectively (the studies are arranged chronologically).
### Table 2.1 Summary of two-dimensional numerical studies on piles subjected to lateral soil movements, 1981 to 2001

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<td>Piles near an embankment</td>
<td>Single pile nearby surface loading</td>
<td>Piles near an embankment</td>
<td>Effect of lateral soil movement on subsequent axial capacity</td>
<td>Piled bridge embankment</td>
<td>Piles subjected to lateral soil movements</td>
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<td>Soil</td>
<td>Soft stratum</td>
<td>Cohesionless &amp; cohesive</td>
<td>Cohesive soil</td>
<td>Underconsolidated clay</td>
<td>Cohesive soil</td>
<td>Cohesionless soil</td>
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<td>Modified Cam-clay model</td>
<td>Linear elastic</td>
<td>Linear elastic</td>
<td>Extended Cam-clay</td>
<td>Tresca, modified Cam-clay, linear elastic</td>
<td>Elasto-plastic (Tresca)</td>
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<td>Pile model</td>
<td>Equivalent sheet-pile</td>
<td>Beam element</td>
<td>Equivalent sheet-pile</td>
<td>Solid element</td>
<td>Beam-column element</td>
<td>Rigid element</td>
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<td>Program</td>
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<td>ABAQUS</td>
<td>AFENA</td>
<td>AVPULL</td>
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<td></td>
<td>Piles in liquefaction-induced lateral spread</td>
<td>Piled bridge embankment</td>
<td>Piles near an embankment</td>
<td>Piled bridge abutment</td>
<td>Piles stabilising slope</td>
</tr>
<tr>
<td>Soil</td>
<td>Cohesionless</td>
<td>Cohesive</td>
<td>Cohesionless &amp; cohesive soil</td>
<td>Cohesive</td>
<td>Cohesionless &amp; cohesive soil</td>
</tr>
<tr>
<td>Soil model</td>
<td>Nonlinear elastic</td>
<td>Linear elastic, linearly elastic-perfectly plastic (Tresca), power law</td>
<td>Nonlinear elastic</td>
<td>Strain Dependent Modified Cam-clay (clay) and linear elastic-perfectly plastic (granular)</td>
<td>Mohr-Coulomb</td>
</tr>
<tr>
<td>Pile model</td>
<td>Beam element</td>
<td>Rigid element</td>
<td>Beam element</td>
<td>Equivalent sheet-pile</td>
<td>Beam element</td>
</tr>
<tr>
<td>Program</td>
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<td>CRISP94</td>
<td>BCPILE</td>
<td>CRISP94</td>
<td>FLAC</td>
</tr>
<tr>
<td>Analysis</td>
<td>Plane-strain</td>
<td>Plane-strain</td>
<td>Plane-strain</td>
<td>Plane-strain</td>
<td>Axi-symmetric</td>
</tr>
</tbody>
</table>
Table 2.2 Summary of three-dimensional numerical studies on piles subjected to lateral soil movements, 1983 to 2005

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<tbody>
<tr>
<td>Finite-element analysis of drilled piers used for slope stabilization</td>
<td>Piled bridge embankment</td>
<td>Pile group effect in unstable slope</td>
<td>Piled bridge embankment</td>
<td>Piles subjected to Lateral Soil Movements</td>
<td>Piles stabilising slope</td>
<td>Piles subjected to lateral soil movements</td>
<td></td>
</tr>
<tr>
<td>Soil</td>
<td>N/A</td>
<td>Soft clay, stiff sand</td>
<td>Cohesionless</td>
<td>Cohesive</td>
<td>Cohesive</td>
<td>Cohesive and cohesionless</td>
<td>Cohesive</td>
</tr>
<tr>
<td>Soil model</td>
<td>Elastic &amp; Duncan-Chang's hyperbolic model</td>
<td>Linear elastic</td>
<td>Isotropic elasto-plastic (Drucker-Prager)</td>
<td>Linear elastic (sand) and modified Cam-clay (clay)</td>
<td>Elastic-perfectly-plastic Von Mises model</td>
<td>Mohr-Coulomb</td>
<td>Mohr-Coulomb</td>
</tr>
<tr>
<td>Pile type</td>
<td>Piles in row</td>
<td>A row of free headed piles and a pile group</td>
<td>Pile group</td>
<td>Pile group</td>
<td>Single piles and two pile group</td>
<td>Single piles and pile group</td>
<td>Single pile</td>
</tr>
<tr>
<td>Pile model</td>
<td>Solid element</td>
<td>Solid element</td>
<td>Solid element</td>
<td>Solid element</td>
<td>Solid element</td>
<td>Beam element</td>
<td>Beam element</td>
</tr>
<tr>
<td>Program</td>
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<td>CESAR-LCPC</td>
<td>CRIPS94</td>
<td>ABAQUS</td>
<td>FLAC</td>
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<tr>
<td>Analysis</td>
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<td>Finite element</td>
<td>Finite element</td>
<td>Finite element</td>
<td>Finite element</td>
<td>Finite difference</td>
<td>Finite element</td>
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</tbody>
</table>
2.2 Laboratory model tests on piles subjected to lateral soil movements

At present, there are relatively few published laboratory model pile or full-scale field pile tests on piles subjected to lateral soil movements. The existing tests can be classified into four categories:

1) lateral soil movements induced by the construction of an embankment at the soil surface adjacent to the piles
2) lateral soil movements induced by deep excavation activities adjacent to the piled foundation
3) lateral soil movements induced by an unstable slope
4) lateral soil movements as a result of lateral spreading induced by earthquakes.

The following subsection presents a review of the reported model tests and the design of the testing apparatus used to perform the tests. This review provides insights into, and assists with the design of the new testing apparatus used in the current study. A full description of the testing apparatus is presented in Chapter 3.
2.2.1 Published laboratory model pile tests

Fukuoka (1977) reported upon a model test conducted by Fukumoto (1975). A model pile was installed into a rectangular iron model box filled with soil. The box was moved horizontally by using a hydraulic jack, as shown in Figure 2.7. This movement simulated a uniform distribution of soil displacement along the depth of the pile section embedded in the box. The results from the test indicated that the deformed shape of the piles was dependent upon the flexural stiffness of the pile.

A number of years later Matsui et al. (1982) carried out a series of model tests on circular piles in a row to validate the theoretical equations presented earlier by Ito and Matsui (1975). Figure 2.8 presents the schematic view of the test equipment used to carry out the model tests. The experimental results justified the assumptions made on the plastic state of the soil for the theoretical derivation. The comparison between the experimental and the theoretical results indicates that a good agreement was attained for the tests carried out on the clay and the sand samples. Based on these comparisons, it was found that the ultimate soil pressure, acting on the passive piles in a row, could be estimated as 1.6 times the theoretical lateral soil pressure, based on the earlier theory of Ito and Matsui (1975), with sufficient confidence over a wide range of pile spacings.
Later studies by Towhata and Al-Hussaini (1988) involved a series of model tests to identify the intensity of the drag force brought about by a mudflow on a solid cylindrical column. In the tests, a model steel rod was dragged along a model tank filled with clay slurry. The test results concluded that the drag force was proportional to both the embedment and the diameter of the column, and that the water content of the clay slurry had a strong effect on the magnitude of the drag force. In contrast, the velocity of the flow had little influence on the drag force.

Springman (1989) performed a series of experiments on the geotechnical centrifuge to investigate the performance of a row of free-headed piles and a pile group in different pile and foundation geometries (the general design of the centrifugal model is shown in Figure 2.9). The focus of the experiment was to study the interaction between the loads applied on the soil surface and the behaviour of the adjacent long vertical piles embedded in a stiff sub-stratum. The influence of the lateral thrust on the piles in a upper soft clay (due to simulated embankment construction) was examined, while soil-pile interaction mechanisms were identified for the behaviour of piles at both working load and ultimate lateral capacity.

Later Barr et al (1991) designed a large shear box used initially to study the performance of soil nails. The shear box was turned 90° compared with the normal shear boxes used in soil mechanics laboratory. The overall inside dimensions of the box were 3 m × 1.5 m × 1.5 m, with the shear plane having an area of 2.25 m². One of the nails tested was instrumented with strain gauges along its shaft, and was located centrally in the shear box filled with sand, with its mid-point on the shear plane. A uniform soil movement was applied to the nail, and the bending moments were recorded. The results revealed that the bending moments were symmetrical about the mid-point, and that the maximum values were near the shear plane.

In the same year, Ginzbury et al (1991) conducted a series of model tests on 2-row, 3-row and 4-row pile groups to investigate the pattern of pressure distribution among piles in different rows subjected to lateral soil movement. Strain gauges were installed on the piles, which were placed in sand or loam deposits underlaid with dense clay; the pile tip was assumed to be fixed elastically. The pile head was either free or
capped by a slab. The uniform soil movements were applied in increments to the piles; the bending moments in piles at different locations within a group were recorded. The soil pressures acting on the pile were derived from the measured bending moments. These pressures were not uniformly distributed among the free-headed piles, however, the pressures were distributed relatively uniformly among the capped piles.

Stewart (1992) undertook a series of centrifuge model tests to obtain detailed data regarding embankment-soil-pile interaction behaviour (the design of the centrifuge model is shown in Figure 2.12). The sand embankment was constructed in stages adjacent to individual piles, and pile groups were installed through a soft clay stratum and into an underlaying dense sand layer. The response of the piles was monitored in terms of their bending moment distribution and pile cap deflection. The undrained response and the longer-term consolidation behaviour were observed. Various pile group configurations, embankment geometry and soil stratigraphy were examined, with the initial tests aimed at modelling a site specific prototype.

Chen (1994) carried out a series of laboratory model tests on instrumented model piles (single and groups) embedded in calcareous sand subjected to a triangular profile of lateral soil movement (the configuration of the experimental apparatus is shown in Figure 2.11). Key parameters, such as pile head fixity condition, the ratio of the depth of moving soil to the pile embedded length; and the pile diameter and stiffness, were identified, and influenced the maximum bending moment of the piles in a constant sand density. The experimental results agreed reasonably well with the theoretical predictions for an existing boundary element program.
Figure 2.8 Model setup (Matsui et al, 1982)
Figure 2.9 Arrangement of centrifuge model (Springman, 1989)

Figure 2.10 Sectional view of model (Anagnostopoulos et al, 1991)
A series of centrifuge tests were conducted by Bransby (1995) to investigate the behaviour of a pile group consisting of two rows of pile, when an adjacent vertical surcharge was applied to the soil (the setup of the test is shown in Figure 2.13). The button-hole foundation behaviour technique was studied by conducting the second of each pair of the tests so that comparisons of the foundation response could be made. The pore pressures were also recorded throughout the centrifuge flight in all tests to determine the degree of consolidation of the clay layer at different time instances, to reproduce the stress path of the soil. The passive lateral soil pressure was found to induce on the button-hole foundation, approximately one-third of the value on the conventional foundation.

Later Pan (1998) performed a series of laboratory model tests on instrumented model piles (single and groups) embedded in soft clay, subjected to a uniform rectangular profile of lateral soil movement (the design of the experimental apparatus is shown in Figure 2.14). The key parameters (such as pile head fixity condition, pile stiffness, and spacing between piles in groups) were identified, and were found to contribute to the distribution of the limiting soil pressure acting along the pile shafts.

Leung et al (2000) undertook a series of centrifuge model tests on unstrutted deep excavation in dense sand, and its influence on an adjacent single pile foundation behind a retaining wall (as shown in Figure 2.15). The condition of the wall (stable or collapse), pile head condition (fixed or free), and the distance from the wall affected the bending moment and deflection of the pile.
Figure 2.11 Overview of experimental apparatus (Chen, 1994)
Chapter 2: Literature review

Figure 2.12 Configuration of centrifuge model (Stewart, 1992)

Figure 2.13 Centrifuge test model (Bransby, 1995)
Figure 2.14 Model test setup (Pan, 1998)

Figure 2.15 Configuration of centrifuge model (Leung, 2000)
A large-scale laminar shear box was developed by Tsuchiya et al (2001) to carry out a series of instrumented piles embedded in silica sand, subjected to various profiles of lateral soil movement (the configuration of the experimental apparatus is shown in Figure 2.16). The influence of the lateral ground movements on the behaviour of a pile and their failure patterns were investigated; the tests indicated that the strain distribution of the pile was clearly affected by the profile of the lateral soil movements. This shear box also has the ability to impose both axial load and lateral soil movement simultaneously to the pile.

![Figure 2.16 Large-scale laminar shear box design (Tsuchiya et al, 2001)](image-url)
### 2.2.1 Summary of the laboratory model tests

A summary of the laboratory model tests is presented in Table 2.3(a), 1974 to 1990 and Table 2.3(b), 1991 to 2003. These studies are arranged chronologically.

Table 2.3(a) Summary of the reported laboratory model scale and full-scale tests on piles subjected to lateral soil movements, 1974 to 1990

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<td>Piles stabilising slope</td>
<td>Piles stabilising slope</td>
<td>Slope stabilising piles in a row</td>
<td>Single piles in marine slide</td>
<td>Piles stabilising landslides</td>
<td>Single piles and pile groups (embankment)</td>
<td>Pile groups in free-head and fixed-head</td>
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<tr>
<td>Type of tests</td>
<td>Model tests</td>
<td>Model tests</td>
<td>Model tests</td>
<td>Model tests</td>
<td>Centrifuge tests</td>
<td>Model tests</td>
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<td>Type of soil</td>
<td>Sand (Ottawa and Blast)</td>
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<td>Three kinds of clay, sand</td>
<td>Clay</td>
<td>Aluminium rods to represent sand</td>
<td>Sand and clay</td>
<td>Sand and loam</td>
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<td>Solid rectangular</td>
<td>Circular bar</td>
<td>Solid circular rod</td>
<td>Solid rectangular and circular</td>
<td>Circular tube</td>
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<tr>
<td>Sensors</td>
<td>Soil pressure cell</td>
<td>Strain gauges (no info on bridge conf.)</td>
<td>External load cells</td>
<td>Strain gauges (force measurements)</td>
<td>Strain gauges</td>
<td>8 half bridge strain gauges</td>
<td>7 strain gauges (no info on bridge conf.)</td>
</tr>
<tr>
<td>Material of pile</td>
<td>Plexiglas</td>
<td>Steel and wooden</td>
<td>N/A</td>
<td>Steel</td>
<td>N/A</td>
<td>Duralumin</td>
<td>Metal</td>
</tr>
<tr>
<td>Dimensions of pile</td>
<td>2.22 cm (d)</td>
<td>5.0 x 0.85cm (steel) and 5.0 x 1.5cm (wooden)</td>
<td>20~40mm (d) and 300mm long</td>
<td>4.7 mm, 9.5 mm (d)</td>
<td>20mm, 30mm, 50 mm (d), 70mm long</td>
<td>12.7mm (d), 300mm long</td>
<td>20 mm (d) and 40 cm long</td>
</tr>
<tr>
<td>Dimensions of tank</td>
<td>15.24cm wide x 60.96 cm long x 30.5 cm deep</td>
<td>10x20x10 (stable layer) cm</td>
<td>60x30x30cm</td>
<td>120cm x 11cm x 13.5cm</td>
<td>100cm x 60cm x 18cm</td>
<td>67.5cm x 20cm x 21cm</td>
<td>N/A</td>
</tr>
<tr>
<td>Shear box</td>
<td>Reinforced plexiglass</td>
<td>Steel</td>
<td>Steel</td>
<td>Steel</td>
<td>N/A</td>
<td>Aluminium</td>
<td>N/A</td>
</tr>
</tbody>
</table>
Table 2.3(b) summary of the reported laboratory model scale and full-scale tests on piles subjected to lateral soil movements 1991 to 2003

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of tests</td>
<td>Model tests</td>
<td>Centrifuge tests</td>
<td>Model tests</td>
<td>Centrifuge tests</td>
<td>Model tests</td>
<td>Centrifuge tests</td>
<td>Model tests</td>
<td>Large scale model tests</td>
<td>Centrifuge tests</td>
</tr>
<tr>
<td>Type of soil</td>
<td>Two kinds of sand</td>
<td>Soft clay and dense sand base</td>
<td>Calcareous sand</td>
<td>Sand and clay</td>
<td>Clay</td>
<td>Clay</td>
<td>Sand</td>
<td>Sand</td>
<td>Sand</td>
</tr>
<tr>
<td>Model pile</td>
<td>Circular tube</td>
<td>Square tube</td>
<td>Circular tube</td>
<td>Circular tube</td>
<td>Solid square strip</td>
<td>Circular tube</td>
<td>Circular tube</td>
<td>Circular tube</td>
<td></td>
</tr>
<tr>
<td>Sensors per pile</td>
<td>Earth pressure panels, which could measure normal and tangential earth pressure</td>
<td>10 half bridge strain gauges</td>
<td>10 full bridge strain gauges</td>
<td>7 full bridge strain gauges</td>
<td>11 full bridge strain gauges on pile and 6 full bridge strain gauges on abutment wall</td>
<td>10 soil pressure cells</td>
<td>20 quarter bridge strain gauges at both side of the piles</td>
<td>2 pore pressure cells, 2 load cells and 2 soil pressure cells</td>
<td>3 soil pressure cells</td>
</tr>
<tr>
<td>Material of pile</td>
<td>N/A</td>
<td>Brass</td>
<td>Aluminium</td>
<td>Duralumin</td>
<td>Duralumin</td>
<td>Aluminium</td>
<td>Steel</td>
<td>Aluminium</td>
<td>Aluminium alloy (Dural)</td>
</tr>
<tr>
<td>Dimensions of pile</td>
<td>47mm, 101mm, 165mm, 216mm, 275mm (d), 800mm long</td>
<td>3.18 x 3.18mm and 244mm long</td>
<td>25<del>50mm (d), 1000mm long, and 1.2</del>2.0mm thick wall</td>
<td>12.7mm (d) and 300mm long</td>
<td>12.7 mm (d) and 190 mm long</td>
<td>295 mm long and 20 mm wide and 2~6mm thick</td>
<td>165.2mm (d), 5.3m long and 3.7 mm thick</td>
<td>20 mm (d) and 450 mm long</td>
<td>9.2<del>9.3mm (d) and 160</del>180mm long</td>
</tr>
<tr>
<td>Dimensions of tank</td>
<td>100x100x175cm</td>
<td>39cm x 65cm x 32.5cm</td>
<td>45 x 56.5cm x 70.0cm</td>
<td>67.5cm x 20cm x 56cm</td>
<td>53.5 cm x 67.5cm x 20cm</td>
<td>57cm x 32.1cm x 21.5cm</td>
<td>2.5m x 2.5m x 8.1m</td>
<td>45cm x 15 cm 25 cm</td>
<td>56 cm x 23.5 cm x 22 cm</td>
</tr>
<tr>
<td>Material of tank</td>
<td>Timber</td>
<td>Aluminium</td>
<td>Steel</td>
<td>Aluminium</td>
<td>Aluminium</td>
<td>Steel</td>
<td>Steel</td>
<td>Aluminium</td>
<td>Aluminium alloy (Dural)</td>
</tr>
</tbody>
</table>
2.3 Pile subjected to combined loadings

Pile foundations designed to support structures and services are often subjected to a series of combined loadings. These loadings can be in the form of axial, lateral and torsional loads, as well as vertical soil and lateral soil movements. Many studies on piles have assessed each of these loadings. However, little information is available for evaluating the response of a combination of these loadings. For example, in rows of either isolated or contiguous piles where commonly used for basement walls, the pressures from the retained soil, as well as the weight from the superstructures are imposed simultaneously onto the piles.

Indeed studies on piles related to liquefaction induced lateral soil movement have found that the axial load on the piles can cause: (1) additional bending moment; (2) additional compression stress; and (3) additional lateral displacement (e.g. Bhattacharya, 2003). On the other hand, lateral soil movement may increase the axial capacity of the piles (Chen et al., 2002). Such an increase is similar to that noted for a laterally loaded pile subjected to axial load from the physical model tests by Anagnostopoulos and Georgiadis (1993), and the numerical simulations by Trochanis et al. (1991).

The focus of the current study is on axially loaded piles subjected to lateral soil movement. Importantly, there are similarities between piles subjected to lateral load (also known as active piles) and piles subjected to lateral soil movement (also known as passive piles). Figure 2.17 shows the similarity between these two groups of piles, in terms of their loadings. It can be seen that, at the surface of the stable soil layer, the equivalent loads imposed on the pile are no different for either the active or the passive piles. With this similarity in mind, it was seen as important to review the past research on axially loaded piles subjected to either lateral load or lateral soil movement, simultaneously. A summary of these studies is tabulated in Table 2.4.
Figure 2.17 Similarity between axially loaded piles subjected to lateral load and lateral soil movement
Table 2.4 Studies on axially loaded piles subjected to either lateral load or lateral soil movement

<table>
<thead>
<tr>
<th></th>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Effect of lateral soil movement on the axial capacity of the piles</td>
<td>Yes</td>
<td>Yes</td>
<td>Effect of lateral loading on the axial pile response and the effect of axial loading on the lateral pile response</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Effect of nonlinear soil behaviour on the axial and lateral response of the piles</td>
<td>No</td>
<td>Yes</td>
<td>Response of piles subjected to liquefaction-induced lateral spread</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>Effect of lateral loading on the axial pile response and the effect of axial loading on the lateral pile response</td>
<td>Yes</td>
<td>No</td>
<td>Effect on the vertical bearing capacity of piles due to lateral soil movement</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Key conclusion drawn</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>The moderate level of soil movement has no significant effect on the axial pile resistance</td>
<td>The lateral load causes rather limited effect on the ultimate axial pile resistance</td>
<td>At large soil movement generated as a result of lateral spread, the level of axial load changes the failure mode of the pile</td>
<td>The axial resistance may increase when pile subjected to soil movement</td>
<td>As pile subjected to lateral soil movement, axial load causes additional bending moment, compression stress and lateral displacement on the pile</td>
<td>In cohesionless soil, the presence of vertical loads increases the lateral load capacity of the pile</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
2.4 Conclusions

A summary of the pressure based methods in which pressure is assumed to be generated onto a pile from moving soil shows that the pressure distribution can be in the form of different shapes. These shapes proposed by different researchers consist mainly of rectangular, triangular and parabolic shapes (Subsection 2.1.1). However, these methods do not provide an estimate of the pile displacement.

On the other hand, displacement methods, where the compatibility between the soil and the pile is satisfied, often require the correct estimate of the free-field displacement profile. A number of solutions currently available assume that the soil is an elastic medium. Although, the solution can be modified to cater for the elastic perfectly plastic soil, the solution can be very complex.

Laboratory studies directly addressing the response of either single piles or pile groups to lateral soil movement have been summarised (see summary in Table 2.3). These studies are generally case-sensitive and insufficient, when compared with the pile failure modes occurring in the field (e.g. Heyman and Boersma, 1961; Heyman, 1965; Leussink and Wenz, 1969; Esu and D'Elia, 1974; Carruba et al 1989, Kalteziotis et al, 1993). For example, field results and a theoretical exploration by Viggani (1981) indicated that, for a rigid free-head pile, six different modes of failure can exist. However, the available laboratory results (Chen, 1994; Barr, 1991) generally reveal only one mode of pile failure.

Additionally, the studies, identified in the literature review, have not addressed the importance of axial compression or tensile pile force to the deflection and the stress response of piles, especially, the increment or the reduction of the pile responses. Overall, a comprehensive experiment is needed to explore: (1) the possible modes of the pile failure derived from the theoretical analysis; (2) the effect of the relative ratio of limiting bearing stress between the moving and the stable layers; (3) the effect of pile-soil relative stiffness on the relative contribution of axial and shear force in the resistance to soil movement; (4) the effect of combined loadings on the pile response; (5) the effect of different shapes of the soil movement profile; and (6) the behaviour of piles in groups.
Chapter 3: Experimental Investigation

3.0 Introduction

As previously concluded in Chapter 2, the limitations of the previous studies of piles subjected to lateral soil movements were concentrated on a number of parameters affecting the pile behaviour or replicated for a certain field condition. However, many uncertainties exist about how these parameters effect the pile behaviour, one of which the axial load, has yet to be investigated. For the current study, with the support of the Australian Research Council, a new apparatus was developed to investigate the response of piles subjected to axial load and lateral soil movement. The apparatus setup, along with the experimental procedure for pile testing in sand, are presented in this chapter, while the results from the tests are categorised and presented in the Chapters 4, 5 and 7, respectively:

1) the single pile tests in sand (Chapter 4)
2) the pile group tests in sand in (Chapter 5)
3) the single piles and the pile group tests in clay (Chapter 7).

An overview chart of the experimental program is shown in Figure 3.1

![Figure 3.1 Presentation chart of experimental work in this thesis](image-url)
3.1 Experimental investigation

The experimental investigation examined the following parameters:

1) axial load applied on the pile head or pile cap for pile group tests (Q);
2) shape of the soil movement profiles (rectangular, triangular and arc) and the magnitude of the movement ($w_s$);
3) ratio of thickness of the moving layer ($L_m$) to the stable layer ($L_s$) ($L_m/L_s = 200/500$ and $400/300$);
4) pile diameter ($d = 25$ mm, $32$ mm, and $50$ mm);
5) strength and deformation properties of sand ($D_s$, $E_s$) and clay ($c_u$);
6) pile arrangement for pile group tests ($1 \times 2$, $2 \times 1$ and $2 \times 2$); and
7) centre to centre pile spacing between piles in pile group tests ($3d$ and $5d$).

A large number of permutations were possible, as can be seen from the summary of parameters to be varied, and the number of variants proposed for each parameter. By investigating previous research studies (Chen, 1994; Pan 1998; Bhattacharya, 2003) and by the careful selection of the parameters, the majority of the tests were undertaken using various parameters (especially 1, 2 and 3 above) which had not been investigated previously. The vast majority of the single and the pile group tests were conducted in sand.

3.2 Apparatus design

A new apparatus was developed for the current study, (previously described in Guo and Ghee, 2004). The apparatus consisted of a laminar shear box, load application equipment, sand/clay, piles, and a data acquisition system (the component details are presented in the following subsections). It is important to note that, in this chapter, the described apparatus was used mainly for single pile tests in sand. For ease of presentation, and to prevent confusion, the modification of the apparatus setup and the experimental procedure for pile group tests in sand, and the single piles/pile group tests in clay, are presented separately in the Chapters 5 and 7, respectively.
3.2.1 Laminar shear box

The laminar shear box apparatus (Figure 3.2), used for a single free-head pile subjected to axial load and lateral soil movement (rectangular profile), simultaneously, comprised of a lower 400 mm height fixed timber box with plan internal dimensions of 1000 mm × 1000 mm. The upper part of the shear box consisted of 25 mm thick laminar aluminium frames, with the same internal dimension as the fixed timber box shown in Figure 3.2(a). These laminar aluminium frames (which were allowed to slide) contained the moving layer (L_m). The timber box, together with a certain number of the laminar frames, were fixed, and hence contained the stable layer (L_s). In this way the soil depths of L_m and L_s were varied between each test, respectively.

Two other configurations were, either the triangular or the arc soil movement profile, was applied to the pile shown in Figures 3.3 and 3.5, respectively.

a) Rectangular soil movement profiles configuration

The rectangular soil movement profile was induced using a rectangular shaped loading block attached onto the head of the hydraulic jack as shown in Figures Figure 3.2(b) and (d). In the rectangular soil movement profile, the soil movement (w_s) was measured from the displacement of the laminar frames of the shear box which formed the moving soil layer (L_m).
Figure 3.2 Experimental apparatus setup

Note: all dimensions in mm
Chapter 3: Experimental investigation

(c) Various components of the experimental apparatus

Refer to Figure 3.2(d) for enlargement

(d) Application of lateral soil movement – rectangular profile

Figure 3.2 Experimental apparatus setup
b) Triangular soil movement profile configuration

The triangular soil movement profile was induced using a triangular shaped loading block, attached to the head of the hydraulic jack (Figure 3.3). The soil movement ($w_s$) was measured from the displacement of the top laminar frame. The top laminar frame of the shear box moved initially; then the lower laminar frames moved as the soil movement $w_s$ increased.

![Figure 3.3 Mechanism of triangular soil movement profile](image)
Since each laminar frame was 25 mm thick, the actual edge of the soil movement profile became a saw-edged pattern rather than a perfect line as indicated in Figures 3.3 and 3.4. The $L_m$ (referred to in the tests with the triangular soil movement profile) was taken as the depth of the moving layer when $w_s$ reached a maximum value (or $>60$ mm). The moving layer may not always reach the expected depth with the lower soil movement ($w_s < 60$ mm), as shown in Figure 3.3(d). Nevertheless, it was important that this mechanism of soil movement be clarified, to allow the current experimental results to be used for future study, or for comparative study, with the results from the model tests carried out by other researchers (Chen, 1994; Tsuchiya et al., 2001). Table 3.1 shows the soil movement profile and the depth of $L_m$ increased with $w_s$.

Table 3.1 Magnitude of soil movement and the corresponding depth of sliding layer for the triangular soil movement profile

<table>
<thead>
<tr>
<th>Soil movement, $w_s$ (mm)</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of moving layer, $L_m$ (mm)</td>
<td>50</td>
<td>75</td>
<td>125</td>
<td>150</td>
<td>175</td>
<td>200</td>
</tr>
<tr>
<td>Soil movement profile</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Soil movement, $w_s$ (mm)</th>
<th>70</th>
<th>80</th>
<th>90</th>
<th>100</th>
<th>110</th>
<th>120</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of moving layer, $L_m$ (mm)</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>Soil movement profile</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 3.4 Application of triangular soil movement profile with $L_m/L_s = 200/500$ at $w_s$ of 10 mm, 20 mm, 70 mm and 150 mm
The arc soil movement profile was induced using an arc-shaped loading block, attached to the head of the hydraulic jack, as shown in Figures 3.5 and 3.6. The soil movement ($w_s$) was measured from the displacement of the top laminar frame of the shear box, which moved initially, followed by the movement of the lower laminar frames, as $w_s$ increased (higher magnitudes of the relative movements between the laminar frames was recorded when compared to the triangular soil movement profile). As each laminar frame was 25 mm thick, the actual edge of the soil movement profile became a saw-edged pattern rather than a perfect line, as indicated in Figure 3.6. The $L_m$, referred to in the tests with arc soil movement profile, was taken as the depth of the moving layer when $w_s$ reached a maximum value (or $>130$ mm). The moving layer may not always reach the expected depth at a lower soil movement ($w_s < 130$ mm), as shown in Figure 3.6(d). Table 3.2 shows the soil movement profile and the depth of $L_m$ increased with $w_s$. 

---

**c) Arc soil movement profile configuration**

The arc soil movement profile was induced using an arc-shaped loading block, attached to the head of the hydraulic jack, as shown in Figures 3.5 and 3.6. The soil movement ($w_s$) was measured from the displacement of the top laminar frame of the shear box, which moved initially, followed by the movement of the lower laminar frames, as $w_s$ increased (higher magnitudes of the relative movements between the laminar frames was recorded when compared to the triangular soil movement profile). As each laminar frame was 25 mm thick, the actual edge of the soil movement profile became a saw-edged pattern rather than a perfect line, as indicated in Figure 3.6. The $L_m$, referred to in the tests with arc soil movement profile, was taken as the depth of the moving layer when $w_s$ reached a maximum value (or $>130$ mm). The moving layer may not always reach the expected depth at a lower soil movement ($w_s < 130$ mm), as shown in Figure 3.6(d). Table 3.2 shows the soil movement profile and the depth of $L_m$ increased with $w_s$. 

---
Chapter 3: Experimental investigation

Figure 3.5 Mechanism of arc soil movement profile

Note: The dimensions of the experimental apparatus are indicated in Figure 3.2, for clarity not repeated in this figure.
Table 3.2 Magnitude of soil movement and the corresponding depth of sliding layer for the arc soil movement profile

<table>
<thead>
<tr>
<th>Soil movement, $w_s$ (mm)</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
<th>70</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of moving layer, $L_m$ (mm)</td>
<td>25</td>
<td>50</td>
<td>50</td>
<td>75</td>
<td>100</td>
<td>100</td>
<td>125</td>
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<tr>
<td>Soil movement profile</td>
<td></td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Soil movement, $w_s$ (mm)</th>
<th>80</th>
<th>90</th>
<th>100</th>
<th>110</th>
<th>120</th>
<th>130</th>
<th>140</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of moving layer, $L_m$ (mm)</td>
<td>125</td>
<td>150</td>
<td>150</td>
<td>175</td>
<td>175</td>
<td>200</td>
<td>200</td>
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<tr>
<td>Soil movement profile</td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>
Figure 3.6 Application of arc soil movement profile with $L_{m}/L_s = 200/500$ of 10 mm to 120 mm, with 10 mm incrementals
3.2.2 Investigation of sand displacement in the shear box

The applied soil movement, \( w_s \), mentioned throughout this thesis, was the displacement magnitude of the laminar frames containing the moving layer of sand \( (L_m) \), as described in Section 3.2.1. The following sections present an overview of the investigation into the displacement of the sand at the surface and within the shear box, respectively.

\textit{a) Sand displacement at the surface}

A transparent plastic sheet (4 mm thick), cut with longitudinal openings of 4 mm (width) and spaced at 40 mm (centre to centre), was used to prepare the grid lines (with coloured sand) on the surface of the shear box. The grid lines enabled a better visual inspection of the sand movement at the surface of the shear box (this method was only implemented at the later stage of the experimental program because having conducted a number of tests, it was found that there were obvious sand moving pattern on the surface of the shear box). Figure 3.7 shows the sand displacement at the surface of the shear box at \( w_s \) of 140 mm. At the vicinity of the pile, the sand displacement contour was complex as the sand particles ‘flow’ around the pile. The magnitude of this displacement contour was not measured due to it was difficult to measure accurately without a proper imaging tool to track the movements of individual sand particles. Except in the vicinity of the pile, the sand displacement, at the left hand side of the boundary way wall, was equal to the laminar frames movement \( (w_s) \). Indeed, the sand displacement gradually reduced to zero (sand at angle of repose) at the right hand side of the boundary wall as its moved away from the soil. Additionally, no measurable variation was found in the magnitude of the sand displacement at the surface between the different applied soil movement profiles.

\textit{b) Sand displacement within the shear box}

For measuring the sand displacement within the shear box, a steel tube of 20 mm in diameter was pushed vertically into the sand bed, by hand. To ensure that the steel tube was inserted vertically, a spirit level was used as a guide during the insertion process. Due to plugging effect, only the bottom part of the steel tube was filled with
sand, hence the tube remained hollow. The coloured sand was then poured into the tube from the top end. Once the tube was completely filled, the tube was extracted slowly and carefully. The initial locations of the coloured sand are shown in Figure 3.9. After the completion of the test, the sand was carefully removed, and both the locations of the coloured sand were measured (Figure 3.10). This procedure to measure the sand displacement within the shear box was only carried out on the tests conducted with $L_m$ of 200 mm. This limitation was due to the difficulty encountered during excavation and measuring the coloured sand, which often unnoticeable or smeared when it was not done (excavation) carefully by hand.

![Figure 3.7](image1.png)  
A) Front section of the shear box  
B) Back section of the shear box

Figure 3.7 The sand displacement at the surface of the shear box at $w_s$ of 140 mm

![Figure 3.8](image2.png)  
Figure 3.8 Typical sand displacement contour at the surface of the shear box
Figure 3.9 Initial locations of coloured sand placed within the shear box

Figure 3.10 Coloured sand recovered at approximately 100 mm depth below the surface and measured
Two assumptions had been made in relation to the coloured sand used to investigate the sand displacement within the shear box:

1) at the completion of the test, if the coloured sand could recovered and measured (distance from the boundary of the shear box and the depth from the surface), it was expected to give an indication of the displacement of the sand within the shear box; and

2) at the completion of the test, if the coloured sand was not recoverable (due to mixing with the other non-coloured sand), it was expected that the coloured sand particles would be placed within the shear zone area (where sand particles are subjected to translation and/or rotation during shearing, as the sand particles try to move past one another, rather than all the particles being in constant translation within the moving soil).

From the tests on all the soil movement profiles (rectangular, triangular and arc), the sand displacements at the surface were generally similar (Figure 3.8). However, as shown in Figures 3.11(a), (b) and (c), the sand displacements within the shear box were varied for the different soil movement profiles.

The tests conducted with the rectangular soil movement profile, the coloured sand at the either front or the back of the pile were unnoticeable at a depth of 125 mm and 170 mm, respectively, thus indicating the possible area of the shear zone of sand in the shear box. This zone was difficult to assess using two strips of coloured sand (Figure 3.9). Hence the extent of the zone can be only an approximation. Nevertheless, the interface between the moving and the stable layers consisted of a zone with a finite thickness rather than a well defined interface. Consequently, this zone was found in the direct shear box test, as previously noted by Atkinson and Bransby (1978).

In the tests conducted with the triangular and the arc soil movement profiles, the shear zones were quite different when compared to the test results with the rectangular soil movement profile. Due to the relative movement between the laminar frames, the generated shear zone extended to the sand surface. Indeed, the arc soil movement profile created the highest relative movement between the laminar frames (with same
\( w_s \); this in turn created the shear zone, which was relatively larger when compared to that created by the triangular soil movement profile.
Figure 3.11 Some indication of the extent of shear zone within the shear box for tests with $L_m/L_s = 200/500$ mm and at $w_s > 60$ mm

(a) Shear zone for the tests with the rectangular soil movement profile

(b) Shear zone for the tests with the triangular soil movement profile

(c) Shear zone for the tests with the arc soil movement profile

Note: All dimensions are in mm

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Chapter 3: *Experimental investigation*
3.2.3 Load application equipment

As shown in Figure 3.12, the hydraulic systems were mounted on a load reaction frame made of four numbers of 75 mm × 125 mm c-channels, each welded to a square end plate of 20 mm thick. The channels were placed vertically and fastened to the laboratory strong floor with 32 mm bolts. These channels (forming vertical columns of the reaction frame) were arranged at four corners of a 1.5 m x 1.0 m square area. Two bridges were made of c-channels and flat plates mounted horizontally across two of the vertical columns. The horizontal (Figure 3.1(c)) and the vertical (Figure 3.2(d)) hydraulic jacks were secured to these bridges, respectively.

Figure 3.12 Load reaction frame
The load applications were simulated by two independent hydraulic systems:

1) the vertical jacking system, (an ENERPAC P-392 hand pump, a V-66 load holding valve, and a RC-1010 hydraulic jack); and

2) the horizontal soil movement system, (an ENERPAC PARG-1100S air hydraulic pump, a V-82 flow control valve, and a RC-1010 hydraulic jack).

3.2.3.1 Pile driving with the vertical jacking system

In all tests, the sand was placed into the shear box before the hydraulic jack was used to drive the pile into the shear box. Prior to the driving process, the pile was initially pushed into the sand bed, by hand, to approximately 100 mm depth. The pile was left standing, and the verticality of the pile was checked and adjusted with a ‘leveller’. Subsequently, the hydraulic jack was secured to the pile head and the driving process commenced; this configuration is shown in Figure 3.13. The hydraulic jack had a maximum length of 300 mm, hence extension rods were used to extend the maximum length of the jack. Every time an extension rod was connected, the pile verticality was checked and adjusted if necessary.

Figure 3.13 Vertical hydraulic jack and connection to a model pile
3.2.3.2 Application of axial load

The axial load was applied by placing each weight block of 10 kg (98.1 N) directly on the pile head. The pile head was located 500 mm above the sand surface (total pile length of 1200 mm). Figure 3.14 shows three 10 kg weights with an equivalent axial load of \( Q = 294 \, \text{N} \) applied on the pile head. This method of load application had no restraint provided on the pile head, hence simulating a free-head pile condition.

Figure 3.14 Axial load applied on the pile head by placing weight blocks of 10 kg each (98.1 N)
3.2.3.3 The horizontal soil movement system

To simulate lateral soil movements, a hydraulic jack and a loading block (Figure 3.15) were used to apply movements to the laminar frames forming the top section of the shear box as shown in Figure 3.1(d). The rate of the lateral movements of the laminar frames and the soil were controlled with the flow control valve. Different movement profiles were imposed (namely arc, triangular and rectangular) by changing the loading block attached to the head of the hydraulic jack, as shown in Figure 3.16.

![Loading blocks to simulate different movement profiles](image)

Figure 3.15 Loading blocks to simulate different movement profiles

![Dimensions of the loading blocks to simulate different soil movement profiles](image)

Figure 3.16 Dimensions of the loading blocks to simulate different soil movement profiles
3.2.4 Sand properties and preparation

The model tests (reported in this chapter) were performed using oven-dried medium grained quartz sand from the Logan River, Carbrook, Queensland. The sand was poorly graded with no fines, was classified as SP by the Australian Standard (AS 1726, 1993). The particle size distribution curve, shown in Figure 3.17, gives an uniformity coefficient $C_u = 2.92$, and a coefficient of curvature $C_c = 1.15$.

![Figure 3.17 Particle size distribution](image)

To reproduce homogeneous sand beds over a wide range of predetermined densities, the sand placement method was particularly important. Indeed, to produce reliable results in the laboratory testing required a consistent density of the sand for each test, especially as small-scale models are sensitive to variations in density. Over the years, many techniques have been developed to prepare sand beds for laboratory testing. Two of the most commonly used methods are:

1) controlling the sand density during the deposition, such as raining techniques (Kolbszewski and Jones, 1961; Hanna, 1963; Walker and Whitaker, 1967; Vesic, 1969; Beiganousky and Marcuson, 1976); and

2) adjusting the sand density after deposition, such as rodding, compacting and vibrating (e.g. Hanna, 1963; Kubo, 1965; Vesic, 1969; Tejhman, 1973).

The raining technique, in which the control of the sand density was dependent upon the intensity and velocity of deposition of the sand grains, was found to be the most
suitable for the present study. The method is simple to adopt, and forms the most uniform and reproducible sand beds, while also being able to reproduce a greater range of sand densities.

A sand rainer was fabricated, for the current study, from timber pieces (with a plan internal dimensions the same as for the shear box) and a height of 150 mm (a photograph of the rainer is presented in Figure 3.18). The rainer was suspended over the shear box by four slings, which were hooked into the four corners of the rainer and were connected to an overhead crane (already located in the laboratory). The crane can be used to raise and lower the sand rainer using a guide frame fabricated from 50 mm $\times$ 50 mm, equal angle aluminium sections bolted to the four corners of the shear box. The laminar frames (Figure 3.2(b)) were placed in stages to allow the rainer to be lowered and raised to the predetermined ‘height of fall’ for the sand to be deposited into the shear box. The base of the sand rainer was made of a fixed timber piece 18 mm thick, overlaying a moveable 6 mm thick plastic piece. Both base pieces were perforated with 5 mm diameter holes in a 35mm $\times$ 50 mm grid pattern. The lower moveable plate was translated by a set of levers located on one side of the rainer. When the holes in the two base plates coincided, the sand was deposited at a controlled intensity over the entire area of the sand box. The velocity of the raining sand was controlled by varying the height of fall, which was maintained at a constant height by raising the sand rainer with the overhead crane at the same rate as the height of sand layer increased. The density of sand was varied from test to test by selecting the height of fall.
A number of sand raining tests were performed to determine the relationship between the dry density ($\gamma_d$) and the height of fall. The results of these calibration tests were plotted and are shown in Figure 3.19.

In these tests the sand fell in discrete streams, which became more diffuse as the height of the fall increased. The density of the sand also increased, from 15.89 kN/m$^3$ to 16.27 kN/m$^3$, as the height of the fall increased, from 400 to 600 mm. The vast majority of the model tests were conducted at a fall height of 600 mm, giving a sand density of 16.27 kN/m$^3$ ($D_r = 0.89$, determined from the relative density test). At this
density, the corresponding friction angle and average Young’s modulus of sand was 38° and 572 kPa, respectively (the tests conducted to obtain these sand properties can be found in Appendix A).

A number of calibration tests were carried out to examine the reproducibility and uniformity of the sand beds produced by the rainer. From the results of these tests, the variation in density between tests, using the same height of fall, was less than 0.1 kN/m³. The uniformity of the sand was checked by raining the sand into eight 101.6 mm diameter moulds, placed in different areas, but at the same height within the shear box. The tests were performed using different height of fall and intensity of deposition elements. The results showed a variation in density of less than 0.1 kN/m³ within the shear box for each height of fall. The particle size distribution tests were conducted from time to time throughout the testing program to monitor any significant changes in the sand grading.

3.2.5 Model pile

Three aluminium model piles, having the same length but different diameters were designed and manufactured for the single pile and pile group tests.

Figure 3.20 shows the locations of the strain gauges on each pile, which was instrumented with ten pairs of strain gauges (a) attached along the shaft to measure the bending strain. Another five pairs of strain gauges (b) were mounted along the shaft at 90° from the bending axis to measure the axial tension or the axial compression strain (only used when there were additional channels available on the data acquisition system). These strain gauges had a gauge resistance of 120 ohm and were excited with 3V (E) bridge voltage. It should be noted that the axial strain can be computed from a pair of ‘a’ strain gauges. Table 3.3 shows the structural properties of the piles.
Chapter 3: Experimental investigation

Figure 3.20 Instrumented model pile

Table 3.3 Structural properties of the instrumented model piles

<table>
<thead>
<tr>
<th>Pile</th>
<th>Length (mm)</th>
<th>Outer Diameter (mm)</th>
<th>Wall Thickness (mm)</th>
<th>Bending Stiffness $E_pI_p$ (kNmm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1200</td>
<td>25.0</td>
<td>1.50</td>
<td>$0.58 \times 10^6$</td>
</tr>
<tr>
<td>2</td>
<td>1200</td>
<td>32.0</td>
<td>1.50</td>
<td>$1.28 \times 10^6$</td>
</tr>
<tr>
<td>3</td>
<td>1200</td>
<td>50.0</td>
<td>2.00</td>
<td>$5.89 \times 10^6$</td>
</tr>
</tbody>
</table>

All the piles were calibrated before the testing commenced. The calibration were carried out with the pile being placed in a simply supported beam arrangement, where both ends of the pile were clamped to the knife-edge supports, spaced at 1000 mm. Known loads were applied at the middle length; and the strain gauges were used to measure the bending moment at each section of the pile. The measured strain at each section was compared with the actual calculated bending moment. This comparison provided a direct calibration between the change in voltage and the bending stress at the strain gauge location.
3.2.6 Data acquisition system

The National Instruments™ data acquisition system (DAQ) was used to acquire both the strain and displacement data from the test. The DAQ consisted of three modules of strain conditioning and one module of linear variable displacement transducer (LVDT) conditioning. Each module of the strain conditioning consisted of eight (8) channels that could be configured in a quarter-bridge, half-bridge or full-bridge configuration of a Wheatstone bridge. The quarter-bridge configuration was used to connect each strain gauge attached to the model pile and to measure, mainly, the bending strain; the half-bridge configuration was used to connect to each strain gauge to measure the axial strain (Figure 3.20). An extra dummy strain gauge was installed to a piece of aluminium plate (quarter-bridge configuration); it was used to record any possible changes in strain due to temperature fluctuations during the duration of the test. Any changes of strain recorded from the dummy gauge were then offset against the other ten recorded strain data for temperature compensation.

To measure the deflection at the pile head, three LVDTs were used together, with the LVDT conditioning module in the DAQ. Two LVDTs were placed horizontally to measure the horizontal displacements at two separate points on the pile head. The rotational angle of the pile head was computed from the measurement of these LVDTs. When the LVDTs were not available (to allow for more channels for the strain gauges), the mechanical dial gauges were used as replacements.

A LabVIEW program, called ‘GEH Project Met.vi’ (VI) was written to control the DAQ, so that the function of null offset, shunt calibration and recording data could be done automatically using the VI. Figure 3.21 presents the graphical interface of the VI. A full control of the DAQ could be undertaken using the control buttons on the screen.
Chapter 3: *Experimental investigation*

The VI program provided a more efficient way of recording data, as the data (strain, load and displacement) were recorded automatically and then saved to a .txt file. The front panel also provided real-time measurements of all channels (strain gauges, load cell and LVDT) to ensure that all channels were functioning during the test. The VI could be programmed to analyse and present the data in the form of the bending moment, shear force, deflection and stress. However, in this study, all the data were analysed separately with an Excel spreadsheet.

Due to the limited number of available displacement transducers and the difficulties encountered in connecting the transducers to each of the laminar frames, an alternative method of measuring the displacement of the top laminar frame was adopted, as shown in Figure 3.22. A piece of graph paper, attached to an independent free-standing frame, was placed adjacent to the shear box. When the laminar frames moved forward (laterally), the displacements were marked on the graph paper. An accuracy of ±0.5 mm could be achieved by using a scale ruler to measure the displacements marked on the graph paper.
3.3 Experimental procedure

The same basic procedure was used for each model pile test. The following outlined the sequence of the experimental procedure:

1) Sand bed preparation

The sand box was first moved into a position beneath the sand raining device, then it was secured (Figure 3.18). The sand was rained at a predetermined height of fall. To maintain a constant height of fall, as the sand bed thickness increased, the height of the rainer was constantly raised. After the raining operation was completed, the shear box was carefully moved by a pallet truck and secured under the loading frame.

2) Driving of pile into the shear box

The vertical hydraulic jack system, as shown in Figure 3.13, was used to drive the pile into the shear box. Prior to the driving process, the pile was initially pushed into the sand bed, by hand, to approximately 100 mm depth. At this instance, the pile was left
standing, and the verticality of the pile was checked and adjusted with a ‘leveller’. The hydraulic jack was secured on the pile head and the driving process commenced. The pressure on the hydraulic jack was recorded at every 100 mm increments in the pile embedment. The hydraulic jack had a maximum extension length of 300 mm, and extension rods were used to increase its extension length. The jacking pressure was released every time an extension rod was connected. However, before the jacking process continued, the pile verticality was checked and adjusted, if necessary.

3) Application of axial load

The axial load was applied on the pile by placing weight blocks, of 10 kg (98 N) each, directly on the pile head, as described in Section 3.2.3.2

4) Setting up instrumentation and data acquisition

The loading devices (Figure 3.2(b)), load cells and LVDTs were then connected to the pile. The input channels from the electronic measuring instruments were connected and initialised. The testing information was keyed into the data acquisition control panel (Figure 3.21). Once the pile test commenced, the data recording process was controlled entirely by the data acquisition system. The data were recorded after every 10 mm lateral soil movement was imposed on the pile via the laminar frames. Up to a maximum $w_s = 170$ mm was applied in each test.

5) Application of lateral soil movement

To simulate the lateral soil movement, the horizontal hydraulic system as described in Section 3.2.3.3, was used. In the single pile tests, either the rectangular, the triangular or the arc soil movement profile was imposed on the pile, as described in Section 3.2.1. The pressure of the horizontal hydraulic jack was recorded once the soil movement was applied.
6) Investigation of sand displacement

When the test was completed, the loading apparatus and the measuring devices were removed. The sand displacement on the surface, and within the shear box, was investigated (as described in Section 3.2.2).

7) End of experiment

At the end of the test, the pile was withdrawn, and the shear box was lifted by the overhead crane and suspended over the storage box. The sand was then emptied from the shear box through an outlet at the base.
3.3.1 Data analysis and processing

The data obtained from the strain gauges were converted to bending moments and plotted to a series of bending moment profiles at different magnitudes of the lateral soil movements. In order to derive the other pile responses, the bending moments and the displacement measurements were then subjected to extensive analysis and data processing. One approach involved fitting the profile to a best-fit polynomial curve, which ranged from the 4th to 7th order, to obtain a continuous distribution of the bending moment profile, as a function of the pile length. Such an approach was used successfully by Springman (1989), Steward (1992) and Chen (1994) to analyse piles subjected to lateral soil movements.

However, the polynomial curve fitting was not always accurate in analysing the pile test data. A number of attempts, however, to fit the current experimental data indicated that the same data could be fitted reasonably well with a number of the polynomial curves of different orders. As a result of this inconsistency, a series of soil reaction profiles, derived for different lateral soil movement magnitudes can be very different in shape; and in some instances, a sudden increase in the magnitude of the soil reaction was noted in the vicinity of the pile tip. Steward (1992) and Springman (1989) in their studies of piles subjected to adjacent embankment construction also affected by this problem. Thus, to address this limitation the current study, numerical differentiation (the finite difference method) was used instead.

As shown in Figure 3.23, by differentiating the bending moment profile to the 1st and 2nd order, the shear force and the soil reaction can be obtained, respectively. On the other hand, by integrating the bending moment profile to the 1st order, the pile rotation can be obtained. Subsequently, the pile rotation profile is integrated, in turn, to obtain the pile deflection.
Figure 3.23 Derivation of other pile responses from the measured bending moment profile

3.3.1.1 Numerical differentiation

In order for the finite difference method to be applied easily to the pile, the strain gauges were spaced at equal lengths, $\Delta z$, as shown in Figure 3.24. The strain gauge measurement, in terms of the bending strain of the beam, was measured at each point of the pile. A series of bending strain, against depth, $z$, were plotted for each loading increment, in this case the applied soil movement ($w_s$).

Figure 3.24 The locations of the strain gauges on the pile
Consequently, the shear force \( s_i \) could be obtained by differentiating the bending moment \( m_i \). This differentiation was achieved by using the 1\(^{\text{st}} \) order finite differentiation relation with Equation 3.1.

\[
s_i = \frac{1}{2} \frac{m_{i-1} - m_{i+1}}{\Delta z}
\]

Equation 3.1

Where \( \Delta z \) is the subinterval for dividing the pile length, and \( m_i \) is the bending moment at a section on the pile.

The soil reaction could be obtained by the 2\(^{\text{nd}} \) order finite differentiation relation with Equation 3.2. As noted by Levachev (2002), this method offered a more reliable value for soil reaction and exact results, when compared to the usual finite difference method.

\[
r_i = \frac{1}{7} \frac{2m_{i-2} - m_{i-1} - 2m_i - m_{i+1} + 2m_{i+2}}{\Delta z^2}
\]

Equation 3.2

Equation 3.2 required five measured bending moments to derive the soil reaction \( r_i \) at a point (location on the pile). Thus, \( r_2 \) to \( r_8 \) could be derived easily from five bending moments measured directly from the pile. However, to derive the soil reaction at \( r_{10} \), the imaginary bending moments, \( m_{-10} \) and \( m_{-9} \), had to be predetermined. With the known boundary conditions at the pile tip, \( m_{-10} \) could be calculated with the method described by Scott (1981). One of the boundary condition was the shear force at the pile tip \( (s_{tip}) \) being equal to zero, with the pile tip having the following characteristics:

\[
\begin{align*}
s_i &= s_{tip} = 0 \quad \text{(boundary condition)} \\
m_{i+1} &= m_{10} \quad \text{(only unknown)} \\
m_{i-1} &= m_{10} \quad \text{(measured from strain gauge)} \\
\Delta z &= 50 \quad \text{(spacing between two points)}
\end{align*}
\]

By substituting the above into Equation 3.1, the results becomes:

\[
m_{-10} = m_{10}
\]
Thus, the imaginary bending moment, $m_{-10}$, is equal to $m_{10}$, measured on the pile (note that these points are in the mirror position). Hence by method of extrapolating, gives $m_9 = m_9$.

3.3.1.2 Numerical integration

Numerical integration with the trapezoidal rule was used, in the current study, to integrate the bending moment profile, and so derive the pile rotation and the pile deflection profiles. As shown in Figure 3.25, the pile rotation was derived using the numerical integration of the 1st order with Equation 3.3. Once the pile rotation profile was obtained, it was further integrated to derive the pile deflection with Equation 3.4.

The integration constants, which consisted of the pile rotation ($\theta_0$) and the pile deflection ($\delta_0$), at the location of $m_0$, were derived (linear extrapolation to obtain the $\delta_0$ and $\theta_0$ at the soil surface) directly from the displacement measured from the LVDTs mounted at the pile head, as shown in Figure 3.2(a). The lateral deflection profiles (at different $w_s$) were relatively accurate and similar in shape, given the integration process (as compared to the differentiation process) involved in deriving them.

\[
\theta_i = \sum_{i=0}^{n} \frac{m_i + m_{i-1}}{2} \Delta z - \theta_0
\]

Equation 3.3

\[
\delta_i = \sum_{i=0}^{n} \frac{\theta_i + \theta_{i-1}}{2} \Delta z - n\Delta z \cdot \theta_0 + \delta_0
\]

Equation 3.4

Figure 3.25 Derivation of the pile rotation and the pile deflection from the measured bending moment
3.4 Limitations and advantages of small-scale model tests

Limitations

The ability of small-scale model tests (or the 1-g model) is well known to have difficulty in modelling the prototype behaviour (or full-scale). A number of factors prevent the results of the small-scale model being extrapolated to any prototype size by using dimensional analysis only. These factors have been outlined by Vesic (1965) as follows:

1) there are significant differences in the nature of the shear phenomenon in sand for different stress ranges. As an example, at a low stress level, the shearing deformation of the sand exhibits volumetric expansion due to the dilation nature of the sand. On the other hand, at a higher stress level, the shearing deformation of the same sand occurs mainly through particle breakage of the sand. In terms of the Mohr-Coulomb failure criterion, it can be said that the dilation and friction angle of the sand depends on the stress level

2) the scaling law lies in the way where the shearing resistance of the sand supporting the pile foundation is established. It is known that, for axially loaded piles, the displacements needed to develop the ultimate load along the pile shaft (skin friction) are dependent on the pile size, while the displacements needed to reach the ultimate load, at the pile tip, are found to be roughly proportionally to the pile diameter

3) the extent of the arching effect on the pile depends on the pile diameter and cannot be assessed directly by dimensional analysis

4) the relative compressibility of sand with pressure that is, the stiffness properties of the sand, depend on the stress level (or pressure).

Advantages

Despite the limitations, small-scale model tests have been commonly used to investigate the behaviour of soil-structure interaction problems in geotechnics. Due to the scale of the model, the effects of important variables can be isolated and studied, in a cost effective manner, to provide qualitative information on the soil-structure interaction problem. Importantly, the cost of model tests is much lower than the cost
of full scale tests. Another advantage of model tests are that they can be conducted in a laboratory, the soil properties can be prepared with consistency, and significant data can be measured more easily than it would be in the field.

3.5 Intended use of model test results in this thesis

The results from the model tests were used in the following manners to achieve the objectives set out for the current research project:

1) to provide good qualitative data where the effect of various parameters effecting the pile can be indentified

2) to provide a basis for comparison between the model test result and the numerical analysis

3) to provide data for the case study and development of the simplified solution.

3.6 Recommendations for future work and use of model test results

Having studied the way a range of input parameters had effect on the pile behaviour with the model tests and numerical analyses. It would be helpful to calibrate the numerical analysis with available full-scale test data, as well as investigate the effect of the strength and stiffness properties at high stress levels. In this way, the numerical analysis, having been able to predict the model tests (at low stress level), can be used to identify the effects of strength and stiffness at high stress level. Consequently, the model tests can be used, with confidence, to predict the pile behaviour in full-scale.

3.7 Summary

The experimental apparatus and the procedure adopted for conducting the model tests on piles, subjected to axial load and lateral soil movement, have been described in this chapter. Throughout the experimental program, the apparatus and the procedure were constantly improved. One of these improvements, only implemented at the latter stage of the experimental program, was the investigation of sand displacement at the surface of, and within, the shear box with the coloured sand. The pile response, in term of strain and displacement, were measured directly using the data acquisition system. From these measurements, the finite difference method was used to derive the shear force and the soil reaction, while the numerical integration was used to derive the pile
rotation and the pile deflection. The results from the model tests are presented and discussed in the following chapters.
Chapter 4: Single Pile Tests

4.0 Introduction of single pile tests

In Chapter 2, a comprehensive literature review highlighted that there are many studies on piles subjected to lateral soil movement. However, each study was unique and focused on a particular area of interest. Therefore, the studies can be categorised into five groups: (1) pile used for slope stabilisation; (2) pile adjacent to excavation; (3) pile adjacent to tunnelling activities; (4) pile subjected to earthquake induced lateral spreading; and (5) pile adjacent to an embankment. Importantly, in many instances, these piles might be subjected to lateral soil movement and axial load, simultaneously. Thus, this chapter presents an overview of the investigation into the behaviour of single piles subjected to these simultaneous loadings. A series of model tests were been conducted with a set of pile and soil parameters at different axial load levels. Figure 4.1 shows the flowchart outlining the chapter’s structure of the presentation of these model tests.
Chapter 4: Single pile tests

Figure 4.1 Flowchart showing presentation of single pile tests in this chapter
4.1 Sign convention

Figure 4.2 shows the sign convention used to interpret the pile response. A downward direction (gravitational direction) describes a positive applied axial load on the pile head. Both the shaft and the bearing resistances are positive in the upward direction, while positive bending moment is taken on the compression side of the pile (back side). Further, positive shear force, computed from the top of the pile, will have an anticlockwise sense of rotation about a point inside an element of the pile. The positive soil movement and the pile deflection are those that act in the same direction of the hydraulic jack movement. On the other hand, a negative lateral soil reaction/resistance will be acting in the direction opposite to the hydraulic jack movement. Additionally, the depth from the surface of the sliding layer is positive in the downward direction. Thus, the pile rotation will be the differentiation of pile deflection over the pile length, being positive when the pile rotates in a clockwise direction.

Figure 4.2 Single pile sign convention
4.2 Tests for single piles subjected to rectangular soil movement profile

A numbering system was used to uniquely identify each test. The parameters used in the tests became part of the numbering system. For an example, a typical test number and representation of the parameters used in the test is shown in Figure 4.3. The parameters, in sequence, are: 1) the moving \((L_m)\) and the stable \((L_s)\) layer thicknesses; 2) the pile diameter; 3) axial load applied at the top of the pile; 4) the falling height used in the sand bed preparation; and 5) the date when the test was conducted. In this section, the tests involved the rectangular soil movement profile; while Sections 4.3 and 4.4, involved the triangular and arc soil movement profiles, respectively. To more easily identify the different profiles used in the model test, except for the rectangular soil movement profile, the capital letters ‘T’ (triangular soil movement profile) and ‘P’ (arc soil movement profile) have been added, after the date, as part of the test identification number. It should be noted that all the tests conducted in this study having: 1) sand density of 16.27 kN/m\(^3\) unless stated otherwise; and 2) a distance \(b_d\) of 500 mm, measured from the centre of the pile to the wall of the shear box (Figure 4.4(b), except for the study on the boundary effect (Subsection 4.3.1). On the tests with axial load, the load was applied by placing weight blocks directly on the pile head, located 500 mm above the sand surface.

The applied load \((Q)\) is calculated as, mass on the pile (kg) \(\times g\) (9.81 m/s\(^2\)), for example: 60 kg \(\times 9.81\) m/s\(^2\) = 589 N. Also to simplify the identification number kg is presented as k.

Figure 4.3 Test identification number for single piles in sand
In the experiment, a number of tests were also repeated, especially when the pile responded differently from the initial expectation. This procedure also served to confirm the test results and the repeatability of the tests. The difference of measured bending moments on the pile between the repeated tests was less than 15%.

Table 4.1 shows tests conducted with the rectangular soil movement profile. Due to the large amount of test data available, the complete test results obtained from each test are provided in Appendix B, where the pile response is presented in terms of bending moment, shear force, soil reaction, pile rotation and pile deflection. In this chapter only the comparisons, mainly between the bending moment profiles, as well as the pile deflection profiles, are presented and discussed.
Table 4.1 Tests conducted on single piles with rectangular soil movement profile

<table>
<thead>
<tr>
<th>Pile-head condition</th>
<th>Soil moving profile</th>
<th>Pile dia. (mm)</th>
<th>Axial load (N)</th>
<th>Moving / Stable layer ((L_m/L_s))</th>
<th>Test Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free-head</td>
<td>Rectangular (dense sand)</td>
<td>25</td>
<td>0</td>
<td>200/500</td>
<td>2-5_25_0k_600_190805</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>400/300</td>
<td>4-3_25_0k_600_081005</td>
</tr>
<tr>
<td></td>
<td></td>
<td>32</td>
<td>0</td>
<td>200/500</td>
<td>2-5_32_0k_600_170304</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>400/300</td>
<td>4-3_32_0k_600_200904</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>294</td>
<td>200/500</td>
<td>2-5_32_30k_600_260304</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>400/300</td>
<td>4-3_32_30k_600_240904</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>589</td>
<td>400/300</td>
<td>4-3_32_60k_600_241005</td>
</tr>
<tr>
<td></td>
<td>Rectangular (dense sand)</td>
<td>50</td>
<td>0</td>
<td>200/500</td>
<td>2-5_50_0k_600_100904</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>400/300</td>
<td>4-3_50_0k_600_100904</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>294</td>
<td>200/500</td>
<td>2-5_50_30k_600_140904</td>
</tr>
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<td></td>
<td></td>
<td></td>
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<td>400/300</td>
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</tbody>
</table>
4.2.1 Boundary effect on model test

Since the boundary effect is one of the factors that can influence the results of the model tests, four tests were conducted to study the boundary effect of the shear box on the pile response. The 50 mm diameter pile was the largest pile used in the single pile tests; it produced the lowest ratio of the boundary distance to the pile diameter. Hence, the effect of the boundary distance was the most critical as compared to the 25 mm and 32 mm piles. Figures 4.4(a), (b) and (c) shows the plan view of the shear box, and the piles placement in three different locations, respectively. Table 4.2 shows the parameters of four tests conducted on 50 mm diameter pile installed at $b_d$ of 750 mm, 500 mm and 250 mm, which is measured between the centre of the pile to the inner face of the shear box wall.

![Figure 4.4 Location of a model pile in the shear box](image-url)
Table 4.2 Tests conducted to study the boundary effect of the shear box

<table>
<thead>
<tr>
<th>Pile-head condition</th>
<th>Soil movement profile</th>
<th>Distance from boundary, (b_d) (mm)</th>
<th>(L_m/L_s)</th>
<th>Test number</th>
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<td></td>
<td>250</td>
<td>400/300</td>
<td>4-3_50_0k_600_300404</td>
</tr>
</tbody>
</table>

**Boundary effect for tests with** \(L_m/L_s = 200/500\)

The first two tests described in Table 4.2 were conducted to investigate the boundary effect at \(L_m/L_s = 200/500\). Figure 4.5 shows the pile response in term of the bending moment and the soil reaction. The following behaviour characteristics are identified from the pile response:

- \(M_{\text{max}}\) at \(b_d = 500\)mm is higher than that at \(b_d = 750\) mm
- above the sliding depth, the soil pressure developed on the pile is positive, as a result of the soil movement
- the soil reaction above sliding depth (S.D.) reduces as the \(b_d\) increases from 500 mm to 750 mm. This reduction indicates that the location where the pile is placed has a significant effect on the pile response, even at a distance of \(b_d = 750\)mm ( or 15 d).

Thus, as the distance \(b_d\) increases the soil pressure, acting on the pile, reduces. This reduction in soil pressure, in turn, causes the bending moment to be reduced.
Chapter 4: Single pile tests

Figure 4.5 Responses of the piles tested at different locations in the shear box with $L_m/L_s = 200/500$: a) bending moment profiles; and b) soil reaction profiles

*Boundary effect for tests with $L_m/L_s = 400/300*

The last two tests described in Table 4.2 were conducted to investigate the pile response closer to the boundary, with a $b_d$ of 500 mm and 250 mm, respectively. Figure 4.6 shows the pile response of these tests conducted at $L_m/L_s = 400/300$. A number of pile behaviour characteristic are identified:

- the bending moment profiles of both the tests has a double curvature shape
- as $b_d$ decreases $M_{\text{max}}$ increases
- $M_{\text{max}}$ increases with $b_d$
- at $b_d = 250$ mm, the maximum soil reaction imposes on the pile above the sliding depth (S.D.) is higher when compare to the pile placed at $b_d = 500$ mm.
Figure 4.6 Responses of the piles tested at different locations in the shear box with $L_m/L_s = 400/300$: a) bending moment profiles; and b) soil reaction profiles.
4.2.2 Experimental results with $L_m/L_s = 200/500$

In the following Subsections 4.2.1.1 to 4.2.1.5, the effect of sand density, pile diameter and axial load on a pile are investigated from the tests conducted with $L_m$ and $L_s$ of 200 mm and 500 mm, respectively, resulting in the ratio $L_m/L_s = 200/500$. The effects of these parameters on the pile response are compared against the ‘reference’ test (presented below).

Reference test (2-5_32_0k_600_170304, see Table 4.1)

Figure 4.7 presents the pile response in terms of bending moment, shear force, soil reaction, pile rotation and pile deflection for the 32 mm diameter pile, subjected to soil movements from 10 mm to 60 mm.

The following behaviour characteristics are identified from the pile response:

- the maximum bending moment, $M_{\text{max}}$ occurs at a depth of 460 mm, Figure 4.7(a), (at this depth, the shear force is indeed zero, Figure 4.7(b))
- the bending moment, shear force and soil reaction reach maxima and remain constant beyond $w_s$ of 30 mm ($w_s/d=0.94$) as shown in Figures 4.7(a), (b) and (c), respectively
- the soil reaction within the moving soil layer ($L_m$) has an arc shape and reaches its maximum at a depth of approximately 100 mm ($L_m/2$), Figure 4.7(c)
- the pile rotation profile shows the pile behaving as a rigid pile (rotation angle remains positive for the entire pile length, with small differences between the top and bottom section of the pile), Figure 4.7(d)
- the pile rotates about a depth of 190 mm at $w_s = 0$–10 mm, then translates at $w_s = 10$–30 mm, and remains stationary for $w_s = 30$–60 mm, Figure 4.7(e)
- the pile deflection at the surface is 6.6 mm compared to the corresponding $w_s$ of 60 mm, indicating that the soil in $L_m$ flowed around the pile, Figure 4.7(e).
Figure 4.7 The response of the 32 mm diameter pile at $L_{mp}/L_s = 200/500$
4.2.2.1 Density effect on 32 mm diameter pile

In order to study the effect of density on the pile response, when subjected to a rectangular soil movement profile, three tests were performed on the 32 mm diameter pile with different sand densities. The moving and the stable layers of 200 mm and 500 mm, respectively, have been selected for these tests. The sand beds were prepared with falling heights of 400 mm and 600 mm, with corresponding sand densities of 15.89 kN/m³ and 16.27 kN/m³, respectively. To achieve a lower density (<15.89 kN/m³) of sand, the third sand bed was prepared by slowly shovelling the sand into the shear box. This method produces an average sand bed density of 14.90 kN/m³, which corresponds (extrapolated) to a falling height of 200 mm, as in the sand density against falling height plot presented in Chapter 3. Therefore, for convenient, a sand density of 14.90 kN/m³ is identified with a falling height of 200 mm. Due to the current height of the bottom section of the shear box (timber box), the lowest possible sand density that can be achieved with the sand rainer is 15.89 kN/m³ (minimum 400 mm falling height). Figure 4.8(a) shows the measured bending moment distributions at the soil movement ws = 60 mm; the pile bending moment increases with an increase in the sand density. As shown in Figure 4.8(b), the M_{max} increases by 138 %, as the density increases from 14.90 kN/m³ to 15.89 kN/m³. Subsequently, as the density increases further from 15.89 kN/m³ to 16.27 kN/m³, there is a slight increase (17%) in the bending moments. Figure 4.8(c) shows the relative density of the sand plotted against the M_{max}; the M_{max} increases linearly with the density and the relative density of the sand.
Figure 4.8 Results from 32 mm pile tested on different densities: a) bending moment profiles; b) $M_{\text{max}}$ vs. density; and c) $M_{\text{max}}$ vs. relative density

4.2.2.2 Influence of pile diameter

Three piles, with the diameters of 25 mm, 32 mm and 50 mm have been tested to study the influence of pile diameters on the response of piles when subjected to lateral soil movement, with $L_m$ of 200 mm and $L_s$ of 500 mm. Figure 4.9(a) presents the bending moment profiles of the three different diameter piles when subjected to soil movement of $w_s = 60$ mm. The $M_{\text{max}}$ increases with an increase in the pile diameter. The location of the $M_{\text{max}}$ for the 32 mm diameter pile is at a depth of 460 mm, while the 25 mm and 50 mm diameter piles are located at a depth of 360 mm. As the...
Chapter 4: Single pile tests

Spacing between strain gauges is 100 mm, the accuracy of the location is also within ±100 mm. The $M_{\text{max}}$ increases in the following manner: initially, there is a 42% increase as the pile diameter increases from 25 mm to 32 mm (28%); and subsequently, there is a 124% increase as the pile diameter increases from 32 mm to 50 mm (56%). Regardless of the pile diameter, the $M_{\text{max}}$ reaches its peak at $w_s = 20$ mm; any further increases in $w_s$ beyond 20 mm shows little or no increase in the $M_{\text{max}}$ (Figure 4.9(b)).

Figure 4.9 Responses of 25 mm, 32 mm and 50 mm diameter piles at $L_s/L_m = 200/500$: a) bending moment profile; and b) $M_{\text{max}}$ against $w_s$

4.2.2.3 Influence of axial load on 32 mm diameter pile

Two tests were conducted on a 32 mm diameter pile, in which axial loads, $Q = 0$ N and 294 N, were applied on the pile head, respectively. Figure 4.10 shows the bending moment profiles of the pile at the soil movement, $w_s = 60$ mm. It appears that the magnitude of the bending moment is higher along the pile with $Q = 294$ N, with the difference in the $M_{\text{max}}$ being 62%, when compare to the pile with $Q = 0$ N. The increase in the bending moment is a secondary effect (occurring as the pile deflects), due to the axial load on the pile head. This secondary effect developed when the axial load and the deflection of the pile (away from its initial position inline with the vertical axis) produced an additional moment on the pile, also known as the p-delta effect. The additional bending moment produced from the p-delta effect reduces at a lower depth (see Figure 4.10), possibly because: 1) the soil produced sufficient
resistance (shaft and bearing resistances) to resist the axial load; and 2) the deflection of the pile is reduced with an increase in the pile depth.

![Figure 4.10 Responses of 32 mm diameter pile with axial loads applied on the pile head](image)

4.2.2.4 Influence of axial load on 50 mm diameter pile

Four tests were conducted to study the effect of the axial loads on a 50 mm diameter pile. The axial loads (Q) of 196 N, 294 N and 589 N were applied on the pile head by a mean of weight blocks (98.1 N each). To highlight the comparison, Figure 4.11 presents the pile responses from the tests, respectively. At the soil movement of $w_s = 20$ mm the difference in the $M_{\text{max}}$ is only 14% (Figure 4.11(a)). At $w_s = 20$ mm, the pile deflection is small, hence the second order effect, due to the axial load or the p-delta effect, is also small. In this instance, any additional bending moment due to the p-delta effect is not significant. At $w_s = 60$ mm, the bending moment increases as the axial load increases, from $Q = 0$ N to 589 N. The increment of $M_{\text{max}}$ is quite consistent; thus the increment of the $M_{\text{max}}$ is proportional to the increment of the axial load (Figure 4.11(b)).

The relationship between the $M_{\text{max}}$ and $w_s$ as shown in Figure 4.11(c) were:

- the rate of an increase in the $M_{\text{max}}$ increases with the axial load
• for tests $Q = 0$ N, 196 N and 294 N, respectively, the $M_{\text{max}}$ reaches its peak and remains constant (at $w_s > 20$ mm)
• for test $Q = 589$ N, the $M_{\text{max}}$ continues to increase, at a slow rate, possibly due to the p-delta effect.

Figure 4.11 Responses of 50 mm diameter pile with different axial loads applied on the pile head at soil movement of: a) $w_s = 20$ mm; b) $w_s = 60$ mm; and c) $M_{\text{max}}$ against $w_s$
4.2.3 Experimental results with $L_m/L_s = 400/300$

As presented in Subsection 4.2.2, the tests were conducted with the sliding and stable layers of 200 mm and 500 mm, respectively. This section presents the findings of the investigation into the pile response at a different sliding and stable depths $L_m = 400$ mm and $L_s = 300$ mm, respectively, resulting in an increase in the ratio $L_m/L_s = 0.4$ to $L_m/L_s = 1.3$. The ‘reference’ test presented below will be used for comparison with tests conducted with different sand densities, pile diameters and axial loads.

**Reference test (4-3_32_0k_600_200904, see Table 4.1)**

Figure 4.12 presents the five pile responses, namely the bending moment, shear force, soil reaction, pile rotation and pile deflection, for the 32 mm diameter pile subjected to $w_s$ of 10 mm to 60 mm.

The following are identified from the pile responses:

- the $M_{max}$ occurs at a depth of 520 mm, and increases with $w_s$ (Figure 4.12(a))
- the shape of the bending moment profiles is of double curvature (Figure 4.12(a))
- the bending moment, shear force and soil reaction increase with the soil movement for the whole range of $w_s = 0 \sim 60$ mm. (Figures 4.12(a) to (c))

As shown in Figure 4.12(c), the soil reaction at the surface, up to a depth of 100 mm, is acting in the opposite direction of the soil movement (a shear resistance in the opposite direction is also noted), as the pile deflects more than the soil movement.

The pile rotation and deflection profiles indicate that the pile deflects in a rigid mode. As shown in Figure 4.12(e), the pile rotates initially at a depth of 150 mm ($w_s = 0 \sim 20$ mm), and subsequently at a depth of 560 mm ($w_s > 20$ mm).
Figure 4.12 The response of the 32 mm diameter pile at \( L_m/L_s = 400/300 \)
4.2.3.1 Density effect of 50 mm diameter pile

Two tests were conducted with a 50 mm diameter pile to study the effect of the sand density on the pile response using two sand densities of 14.90 kN/m³ ($D_r = 0.27$) and 16.29 kN/m³ ($D_r = 0.89$). As shown in Figure 4.13(a), the bending moment profiles obtained at the soil movement, $w_s = 60$ mm are: of a single curvature shape on test $\gamma = 14.90$ kN/m³ (loose sand); and of a double curvature shape on test $\gamma = 16.29$ kN/m³ (with the negative bending moment within $L_m$ and the positive bending moment within $L_s$). The negative bending moment, obtained from the test with $\gamma = 16.29$ kN/m³ (dense sand), indicates evidence of a restraint provide by the sliding layer ($L_s$) in which the pile is prevented from rotating. This restraint can be clearly seen in the rotation profiles of the pile (Figure 4.13(b)). The restraint in the rotation provided by the denser sliding layer to the pile also reduces the pile deflection, as can be seen in Figure 4.13(c).

Figure 4.13 Responses of 50 mm diameter pile tested at two densities: a) bending moment profiles
4.2.3.2 Influence of pile diameter

In order to investigate the effect of the pile diameter, three tests were conducted using piles with diameters of 25 mm, 32 mm and 50 mm, respectively. As noted earlier (Subsection 4.2.3.1), the denser a sand bed the more the restraint provided on the pile. Such restraint then causes a negative bending moment above the stable layer (Ls). This restraint is noted on all the tests (Figure 4.14(a)). Further, the shapes of the bending moment profiles for all tests have similar double curvatures. As shown in Figure 4.14(b) the $M_{\text{max}}$ and the $M_{\text{rmax}}$ both increase with an increase in the pile diameter ($w_s = 0\text{~}60\text{~mm}$).

Figure 4.14 Bending moment profiles of 25 mm, 32 mm and 50 mm diameter piles ($L_s/L_m = 400/300$)
4.2.3.3 Influence of axial load on 32 mm diameter pile

To study the effect of the axial load on the response of a pile, tests were conducted with three different levels of axial loads (0 N, 294 N and 589 N). Figure 4.15 shows the bending moment response of the pile at soil movement, $w_s = 60$ mm. The $M_{max}$ increases with the axial load, while the increments of $M_{max}$ are quite consistent (49 % ~ 67 %); a similar increment (62 %) is recorded for the tests conducted on $L_m/L_s = 200/500$ (Section 4.2.2.3). These increments due to the additional axial loads on the pile head (294 N and 589 N) impose additional bending moments to the pile, which resulted a positive bending moment along the entire length of the pile (a shift toward the positive side of the figure). The shape of all the bending moment profiles is of double curvature with the location of $M_{max}$ consistently located at the vicinity of 460 mm depth, which is slightly below the sliding layer ($L_s = 400$ mm).

![Figure 4.15 Responses of 32 mm diameter pile tested at three different axial loads (0 N, 294 N and 589 N)](image)

4.2.3.4 Influence of axial load on 50 mm diameter pile

Another three tests were conducted on a 50 mm diameter pile, with axial loads of $Q = 0$ N, 294 N, 589 N being applied to the pile head, respectively. Figure 4.16(a) presents the bending moment profiles at the soil movement, $w_s = 60$ mm. Not surprisingly, the highest value for bending moment is obtained from the test with highest applied axial load (589 N); this is consistent with the tests on the 32 mm diameter pile (see Section...
4.2.3.3). Thus, the bending moment within the $L_m$ changes from negative to positive as the axial load increases. These changes also indicate differences in the deflection profiles of the pile. Interestingly, for the test with 294 N axial load, the value of the $M_{\text{max}}$ (in $L_s$) is the lowest among all the tests. This is mainly due to two net responses: 1) the negative bending moment resulting from the soil movement; and 2) the positive bending moment resulting from the secondary effect (p-delta) caused by the additional axial load.

Figure 4.16(b) shows the relationship between the $M_{\text{max}}$ and the $w_s$ for the tests. It can be seen that:

- the $M_{\text{max}}$ and the $M_{\text{max}}$ for all tests increase with $w_s$
- at $L_m/L_s = 400/300$, the bending moment profile of the test with $Q = 0$ N has a double curvature shape with both the $M_{\text{max}}$ and the $M_{\text{max}}$, however for the tests with $Q = 294$ N and 589 N, only a positive bending moment is recorded. This change in the $M_{\text{max}}$ indicates that the axial load reduces the $M_{\text{max}}$ while it increases the $M_{\text{max}}$
- at $w_s > 90$ mm, the $M_{\text{max}}$ for both tests with an axial load are higher than that test without an axial load
- above $w_s > 90$ mm, the rate of the increase of the $M_{\text{max}}$ reduces, however, the $M_{\text{max}}$ for the tests with $Q = 0$ N and 294 N, respectively, remain constant.

Figure 4.16 Responses of 50 mm diameter pile tested at three different axial loads (0 N, 294 N and 589 N)
4.2.4 Summary of single piles subjected to rectangular soil movement

The following three subsections (4.2.4.1 to 4.2.4.3) summarise the effect of the axial load (Q) and the pile diameter on the $M_{\text{max}}$ for the two ratio of $L_m/L_s$ (200/500 and 400/300).

4.2.4.1 The relationship between the $M_{\text{max}}$ and the pile diameter at different $L_m/L_s$ ratios

Figure 4.17 shows the relationship between the $M_{\text{max}}$ against the pile diameter. The $M_{\text{max}}$ are recorded at $w_s = 60$ mm. It should be noted that the $M_{\text{max}}$ is an absolute maximum value taken from the highest of either the $M_{-\text{max}}$ or the $M_{+\text{max}}$ obtained from a single test.

The findings show that the $M_{\text{max}}$ increases with the pile diameter. However, the rate of the increment is different for the two ratios of $L_m/L_s$. At $L_m/L_s = 400/300$, as the pile diameter increases from 25 mm to 32 mm, the $M_{\text{max}}$ increases by 134 %. The further increase in the pile diameter from 32 mm to 50 mm leads to a smaller increase in the $M_{\text{max}}$ (11.5 %). However, at $L_m/L_s = 200/500$, the increment of the $M_{\text{max}}$ is almost linearly proportional to the increase in the pile diameter.

As previously noted in Subsection 4.2.3.2, for the tests with $L_m/L_s = 400/300$, the bending moment profiles have a double curvature shape with the negative bending moments being within the $L_m$. These negative bending moments are generated from the soil reaction acting in the opposite direction to the soil movement (when the pile deflection surpassing the soil movement). Also, as shown previously in Figure 4.14(a), the $M_{\text{max}}$ increases with the pile diameter, giving a smaller net increase in the $M_{-\text{max}}$ or the $M_{+\text{max}}$. Consequently, the increment of $M_{\text{max}}$ reduces with the pile diameter.

For the $L_m/L_s = 200/500$ tests, the shape of the bending moment profiles are of a single curvature, with only the positive bending moments being recorded. Thus, the $M_{\text{max}}$ increases linearly with the soil reaction (as $w_s$ increases). Similarly, just as a
larger pile diameter leads to a larger force per unit length (soil reaction) acting on the pile, the $M_{\text{max}}$ also increases with the pile diameter.

Figure 4.17 $M_{\text{max}}$ against pile diameter for single pile tests conducted at different ratios of $L_m/L_s$
4.2.4.2 The relationship between $M_{\text{max}}$ and axial load at different ratios of $L_m/L_s$

Figure 4.18 shows the effect of different magnitudes of $Q$ on the $M_{\text{max}}$ of piles subjected to the $w_s$ of 60 mm and 90 mm for $L_m/L_s$ of 200/500 and 400/300, respectively ($w_s = 90$ mm is selected for $L_m/L_s = 400/300$ due to the $M_{\text{max}}$ only reaches its peak at this magnitude, see Figure 4.16(b)). As discussed in the previous Subsections (4.2.2 and 4.2.3), regardless of $Q$, the $M_{\text{max}}$ occurs at a depth in the vicinity of 360 ~ 460 mm and 460 mm ~ 560 mm for the ratio $L_m/L_s$ of 200/500 and 400/300, respectively. These findings show that the $M_{\text{max}}$ is always located below the $L_m$. In general the $M_{\text{max}}$ increases with an increasing $Q$; this increment is almost linear, except for the 50 mm diameter pile on ratio $L_m/L_s = 400/300$.

It can be seen in subsequent Subsection 4.2.4.3 that the pile deflection on the 32 mm diameter pile ($L_m/L_s = 400/300$) is approximately 75 mm (or ~2.3d). Further, in addition to the bending moment induced by the $w_s$, this large deflection, together with $Q$, causes the $M_{\text{max}}$ on the pile to increase more rapidly (Figure 4.18). Thus, for the tests with $L_m/L_s = 400/300$, the $M_{\text{max}}$ tends to increase at a higher rate than that tests with $L_m/L_s = 200/500$.

![Figure 4.18 $M_{\text{max}}$ against axial load for single pile tests conducted at different ratios of $L_m/L_s$](image-url)
4.2.4.3 The relationship between pile deflection and axial load at different ratios of $L_m/L_s$

Figure 4.19 shows the relationship between the pile deflection and Q. The pile deflection is taken at $w_s = 60$ mm. It can be seen as the $L_m/L_s$ changes from 200/500 to 400/300, the pile deflection increases 30 and 9 times for the 32 mm and 50 mm diameter piles, respectively. This increment indicates that both the Q and pile diameter have less of an effect on the pile deflection when compared to the ratio of $L_m/L_s$. For the tests $Q = 0$ N, these differences are as follows: 1) for $L_m/L_s = 200/500$, the pile deflection reduces as the pile diameter increases; and 2) for $L_m/L_s = 400/300$, the pile deflection increases with the pile diameter.

Figure 4.19 $M_{\text{max}}$ against axial load for single pile tests conducted at different ratios of $L_m/L_s$
4.3 Tests for single piles subjected to triangular soil movement profiles

A series of tests was conducted by using the triangular (T) soil movement profile to study the effect of the movement on the pile response. These tests were conducted with sand bed density of 16.27 kN/m³. Table 4.3 shows the tests conducted with the triangular soil movement profile.

Table 4.3 Tests conducted on single piles with the triangular (T) soil movement profile

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<th>Pile-head condition</th>
<th>Soil moving profile</th>
<th>Pile dia. (mm)</th>
<th>Axial load (N)</th>
<th>Moving/ stable layer (Lm/Ls)</th>
<th>Test Number</th>
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<td>400/300</td>
<td>2-5_32_0k_600_071005_T</td>
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<td></td>
<td></td>
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<td>4-3_32_30k_600_111005_T</td>
</tr>
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<td></td>
<td></td>
<td>400/300</td>
<td>4-3_32_30k_600_111005_T</td>
<td></td>
</tr>
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<td></td>
<td></td>
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<td>200/500</td>
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</tbody>
</table>
4.3.1 Experimental results with $L_m/L_s = 200/500$

The following Subsections 4.3.1.1 to 4.3.1.2 outline the effect of the pile diameter and the axial load on piles with $L_m$ and $L_s$ of 200 mm and 500 mm, respectively.

4.3.1.1 Influence of pile diameter

Two tests were conducted on the 32 mm and 50 mm diameter piles to study the influence of the different pile diameters. Figure 4.20(a) presents the pile response in the form of the bending moment at a soil movement of $w_s = 60$ mm. The bending moment obtained for both the 32 mm and 50 mm diameter piles are similar in shape, but different in magnitude. The difference between the $M_{\text{max}}$ is 23 %, which is less than the difference (124 %) obtained from tests subjected to the rectangular soil movement (Section 4.2.2.2). These differences indicate that the bending moment increases with pile diameter (or pile stiffness). As noted previously in Chapter 3, the triangular soil movement profiles gradually increases, and changes from a triangular shape to a trapezoidal shape at $w_s > 80$ mm. This change has significant effect on the rate of mobilisation of bending moment. Figure 4.20(b) presents the $M_{\text{max}}$ against the soil movement plot; the $M_{\text{max}}$ only reaches its peak at $w_s > 80$ mm. The pile deflection profile shown in Figure 4.20(c), indicates that the pile deflection of the 50 mm diameter pile is less that of the 32 mm diameter pile. The difference in the pile deflections, at the surface, is 186 %, which shows that the pile deflection decreases as the pile diameter increases.
Figure 4.20 Responses of 32 mm and 50 mm diameter piles subjected to triangular profile of soil movement: a) bending moment profile; b) $M_{\text{max}}$ against $w_s$; c) pile deflection profile
4.3.1.2 Influence of axial load

(a) On the 32 mm diameter pile

The effect of the axial load on the pile subjected to a triangular soil movement was investigated using two tests: 1) without the axial load; and 2) with $Q = 294$ N, applied on the pile head (700 mm above the sand surface). It can be seen from Figures 4.21(a) and (b) that generally the shapes of the bending moment profiles are similar for both the tests. At $w_s = 60$ mm, it appears that the $M_{\text{max}}$ for the test with $Q = 0$ N is 30% higher than the test with $Q = 294$ N. The lower $M_{\text{max}}$ for the test with $Q = 294$ N can be attributed to the verticality of the pile prior to the application of the soil movement. After the application of the axial load, the pile bends backward, resulting in the initial negative pile deflection, which, in turn, generates a small negative bending moment on the pile (see Figure 4.21(c), at $w_s = 0$–20 mm). Subsequently, as the soil movement is applied to the pile, this negative bending moment is then added to the positive bending moment (resulting from the application of $w_s$) to give a smaller net $M_{\text{max}}$. However, at a higher $w_s$ of 90 mm, the $M_{\text{max}}$ for the test with $Q = 294$ N is larger (by 81.8%), when compare to the test with $Q = 0$ N.

In summary, the pile responses show that the $M_{\text{max}}$ increases with the axial load, although, the initial p-delta effect (due to the verticality of the pile) can cause the $M_{\text{max}}$ to be a negative value even before the application of the $w_s$. 
Figure 4.21 Responses of 32 mm diameter pile subjected to different axial loads and triangular soil movement profile: a) at $w_s = 60$ mm; b) at $w_s = 90$ mm; and c) $M_{\text{max}}$ against $w_s$. 
(b) On the 50 mm diameter pile

The effect of the axial load is further investigated using the larger 50 mm diameter pile. The axial load of 294 N is imposed on the pile head for one test, while the other test has no axial load. The bending moment response of the pile at \( w_s = 60 \) mm is shown in Figure 4.22(a). The \( M_{\text{max}} \) from both tests are located below the moving layer (depths between 400 ~ 500 mm). The difference between the \( M_{\text{max}} \) is 30.7 %. The earlier problem with pile verticality is not evident in these tests. Hence, in these tests, where the pile is vertical before the \( w_s \) is applied, the p-delta effect only generates an additional positive bending moment to the pile (at \( w_s > 0 \) mm).

Figure 4.22(b) shows that the \( M_{\text{max}} \) increases with \( w_s \), up to \( w_s = 80 \) mm. Above \( w_s > 80 \) mm, the \( M_{\text{max}} \) remains constant (or there is only a small increase) for both the tests. The \( M_{\text{max}} \) at the peak (\( w_s = 80 \) mm) of the test with \( Q = 294 \) N is 30.8 % higher than the test with \( Q = 0 \) N.

![Figure 4.22 Responses of 50 mm diameter pile subjected to different axial loads and triangular soil movement profile](image_url)
4.3.2 Experimental results with $L_m/L_s = 400/300$

The following two Subsections present tests conducted with $L_m$ and $L_s$ of 400 mm and 300 mm, respectively, thus having $L_m/L_s$ ratio of 400/300.

4.3.2.1 Influence of pile diameter

Two tests were conducted with the 32 mm and the 50 mm diameter piles, respectively to study the influence of pile diameter on the pile response. Figures 4.23(a) and (b) show the bending moment and the pile deflection profiles, respectively. It can be seen that both $M_{\text{max}}$ and the pile deflection (at the surface) increases with the pile diameter. The pile responses (bending moment and pile deflection profiles) for both the tests are similar in shape. The locations of $M_{\text{max}}$ of these tests are located in the vicinity of 460 mm depth, which is below $L_m$.

![Figure 4.23 Responses of 32 mm and 50 mm diameter piles subjected to triangular soil movement profile: a) bending moment profile; and b) pile deflection profile](image)

Figure 4.23 Responses of 32 mm and 50 mm diameter piles subjected to triangular soil movement profile: a) bending moment profile; and b) pile deflection profile
4.3.2.2 Influence of axial load

(a) Axial load on the 32 mm diameter pile

Two tests investigated the influence of the axial load with Q of 0 N and 294 N, respectively. The pile response of the 32 mm diameter pile, in term of the bending moment at \( w_s = 60 \) mm, is shown in Figure 4.24(a). The location of the maximum bending moments (\( M_{\text{max}} \)) for both tests appear to be at a depth of 460 mm, which is 60 mm below the sliding depth (\( L_m = 400 \) mm). The difference in the \( M_{\text{max}} \) is 183 %. A small magnitude of negative bending moment is found for the test with \( Q = 294 \) N. The finding of this moment could be due to the p-delta effect (a slight deviation in the pile from its vertical position after installation), which occurred after the application of the axial load, but prior to the application of the \( w_s \).

As can be seen in Figure 4.24(b), the \( M_{\text{max}} \) remains low (up to 760 kNm) for both tests at \( w_s < 40 \) mm. At \( w_s > 40 \) mm, a steep increase occurs in \( M_{\text{max}} \). With a higher \( M_{\text{max}} \) being recorded for the test with \( Q = 294 \) N, it can be concluded that the \( M_{\text{max}} \) increases with the axial load \( Q \).

![Figure 4.24 Responses of 32 mm diameter pile subjected to different axial loads (0 N and 294 N)](image-url)
(b) **Axial load on the 50 mm diameter pile**

The effect of the axial load is also investigated for the 50 mm diameter pile with Q of 0 N and 294 N, respectively. The difference in the $M_{\text{max}}$ between the tests is 28% (Figure 4.25(a)). The shapes of the bending moment profiles are of a single curvature, with the $M_{\text{max}}$ located at a depth of 460 mm.

As shown in Figure 4.25(b), in both the tests (Q = 0 N and 294 N), the $M_{\text{max}}$ increases with $w_s$. The $M_{\text{max}}$ obtained from the tests with Q = 294 N is higher than that test with Q = 0 N. However, this difference is insignificant at $w_s < 50$ mm. However, at $w_s > 150$ mm, the $M_{\text{max}}$ has not reached its peak value.

![Graph](image)

**Figure 4.25** Responses of 50 mm diameter pile subjected to different axial loads (0 N and 294 N)
In summary, both the 32 and 50 mm diameter piles show that the $M_{\text{max}}$ increases with axial load ($Q$). Also, though not presented within this chapter, the pile deflection increases with the axial load (see Appendix B).

4.4 Tests for single piles subjected to arc soil movement profiles

A further investigation was carried out with the third soil movement profile type, the arc profile (P). These tests, listed in Table 4.4, were conducted with sand beds having a density of 16.27 kN/m$^3$.

Table 4.4 Tests conducted on single piles with the arc (P) soil movement profile

<table>
<thead>
<tr>
<th>Pile-head condition</th>
<th>Soil moving profile</th>
<th>Pile dia. (mm)</th>
<th>Axial Load (N)</th>
<th>Moving/stable layer ($L_m/L_s$)</th>
<th>Test number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free-head</td>
<td>Arc (dense sand)</td>
<td>32</td>
<td>0</td>
<td>200/500</td>
<td>2-5_32_0k_600_210704_P</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>400/300</td>
<td>4-3_32_0k_600_220704_P</td>
</tr>
<tr>
<td></td>
<td></td>
<td>294</td>
<td>200/500</td>
<td>2-5_32_30k_600_250306_P</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>400/300</td>
<td>4-3_32_30k_600_171005_P</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50</td>
<td>0</td>
<td>200/500</td>
<td>2-5_50_0k_600_140705_P</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>400/300</td>
<td>4-3_50_0k_600_250804_P</td>
</tr>
<tr>
<td></td>
<td></td>
<td>294</td>
<td>200/500</td>
<td>2-5_50_30k_600_150705_P</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>400/300</td>
<td>4-3_50_30k_600_181005_P</td>
</tr>
</tbody>
</table>
4.4.1 Experimental results with $L_m/L_s = 200/500$

The following two Subsections 4.4.1.1 and 4.4.1.2 present an overview of the tests conducted with $L_m$ and $L_s$ of 200 mm and 500 mm, respectively, having a ratio of $L_m/L_s = 200/500$.

4.4.1.1 Influence of pile diameter

Figure 4.26 presents the pile responses of the two tests on the 32 mm and 50 mm diameter piles, respectively. The shape of the bending moment profiles on both tests are similar, while the $M_{\text{max}}$ obtained from the 50 mm diameter pile is approximately five times higher than the $M_{\text{max}}$ obtained from the 32 mm diameter pile. Nevertheless, for both tests, the $M_{\text{max}}$ reach the peak at $w_s = 60$ mm, and remain almost constant between $w_s = 60 \sim 80$mm. Subsequently, the $M_{\text{max}}$ start to increase again as $w_s$ increases beyond 80 mm. This ‘stepping’ increment of the $M_{\text{max}}$ is associated with the application of soil movement of the arc shaped loading block as described in Chapter 3. Indeed, the increase of the $M_{\text{max}}$ is a function of both the $w_s$ and the depth of the sliding layer. In another words, the $M_{\text{max}}$ increases with $w_s$ until the peak is reached, and remains constant for as long as the depth of the sliding layer remains constant (for an arc shape profile, both $w_s$ and the depth of the sliding layer constantly changing, see Table 3.2 in Chapter 3).
Figure 4.26 Responses of 32 mm and 50 mm diameter piles subjected to triangular soil movement profile: a) bending moment profile; and b) $M_{\text{max}}$ against $w_s$.
4.4.1.2 Influence of axial load

(a) Axial load on the 32 mm diameter pile

Figure 4.27(a) shows the pile response for the two tests conducted on the 32 mm diameter pile subjected to soil movement of $w_s = 60$ mm (at the soil surface). The axial loads of $Q$ of 0 N and 294 N were applied to the pile, respectively. The applied $Q = 294$ N causes a negative bending moment at the sand surface (depth = 0 mm), probably due to the pile not being perfectly vertical after it was driven into the sand. The p-delta effect causes the pile to bend backward (initial negative deflection at the pile head), giving the negative bending moment from the sand surface up to a depth of approximately 425 mm. However, at the soil movement of 140 mm, the p-delta effect is offset by the increase of positive pile deflection, as the soil movement increases (Figure 4.27(b)).

Figure 4.27(c) shows an increase in the $M_{\text{max}}$ (depth = 360 mm) with soil movement. For the test with $Q = 0$ N, the $M_{\text{max}}$ reaches its peak at the $w_s = 120$ mm, however, for the test with $Q = 294$ N, the $M_{\text{max}}$ still increases even at $w_s = 150$ mm. Both tests were stopped at the $w_s = 150$ mm. However, if the $w_s$ was to be further increased, it would be expected that for the test with $Q = 294$ N should exceed the peak $M_{\text{max}}$ (40,130 Nmm) obtained from the test with $Q = 0$ N.
Figure 4.27 Responses of 32 mm diameter pile subjected to different axial loads and arc soil movement profile
(b) Axial load on the 50 mm diameter pile

Two tests were conducted to study the effect of the axial load on the 50 mm diameter pile. These tests were conducted with axial load, $Q$ of 0 N and 294 N, respectively. As can be seen in Figure 4.28(a), the $M_{\text{max}}$ (at $w_s = 60$ mm) for the test with $Q = 0$ N is 43\% higher than the test with $Q = 294$ N. At a higher magnitude of soil movement, $w_s > 80$ mm (see Figure 4.28(b)) the test with $Q = 294$ N gives a higher the $M_{\text{max}}$. Both the tests show that: 1) at $w_s > 80$ mm, the $M_{\text{max}}$ reaches its limiting value and remains constant; and 2) at $w_s < 40$ mm, the negative $M_{\text{max}}$ (or $M_{\text{max}}$), in the test with $Q = 294$ N, is likely caused by the pile not being perfectly vertical prior to the application of $w_s$, which, in turn, caused the p-delta effect. However the p-delta effect diminishes as the soil movement and the pile deflection increase (also noted in the above Subsection 4.3.1.2).

In summary, both the 32 mm and the 50 mm diameter pile show that the $M_{\text{max}}$ increases with the axial load, $Q$. 
Figure 4.28 Responses of 50 mm diameter pile subjected to different axial loads and arc soil movement profile: a) bending moment profile; and b) $M_{\text{max}}$ against $w_s$
4.4.2 Experimental results with $L_m/L_s = 400/300$

The following two Subsections 4.4.2.1 and 4.4.2.2 presents tests conducted with $L_m$ and $L_s$ of 400 mm and 300 mm, respectively, thus giving a $L_m/L_s$ ratio of 400/300.

4.4.2.1 Influence of pile diameter

The results of the bending moment profiles of the two tests conducted on the 32 mm and 50 mm diameter piles, respectively, are shown in Figure 4.29. At the soil movement, $w_s = 60$ mm, the difference in the $M_{max}$ for the 32 mm diameter pile, when compared to the 50 mm diameter pile is 39 %. The location of the $M_{max}$ for the 32 mm and the 50 mm diameter piles are 360 mm and 460 mm, respectively. The accuracy of these locations, however, depends on their location of the strain gauges, where they are spaced at 100 mm intervals along the pile. At a depth of more than 500 mm, the difference in the bending moment between the piles is smaller (15 %).

Figure 4.29 Responses of 32 mm and 50 mm diameter piles subjected to arc soil movement profile
4.4.2.2 Influence of axial load

(a) Axial load on the 32 mm diameter pile

Two tests were conducted on a 32 mm diameter pile; axial loads of $Q = 0\ N$ and 289 N were applied to the pile, respectively. Figure 4.30 shows the pile response in terms of the bending moment profile at $w_s = 60\ mm$. The depth of the $M_{\text{max}}$ occurs at the same depth (460 mm) for both the tests. In the tests with $Q = 0\ N$ and 294 N, the $M_{\text{max}}$ are 54,963 Nmm and 106,554 Nmm, respectively. The difference between these $M_{\text{max}}$ is 93.9%.

Figure 4.30(b) shows for the whole range of $w_s$ (0 ~ 110 mm), the $M_{\text{max}}$ from the test with $Q = 294\ N$ is higher than that from the test with $Q = 0\ N$, while, for the test with $Q = 0\ N$, the $M_{\text{max}}$ reaches its peak at $w_s = 120\ mm$.

Figure 4.30 Responses of 32 mm diameter pile subjected to different axial loads and arc soil movement profile
Chapter 4: Single pile tests

(b) Axial load on the 50 mm diameter pile

Figure 4.31 shows the two tests conducted with the same testing parameters as the previous tests, used a 50 mm diameter pile instead of a 32 mm diameter pile. The $M_{\text{max}}$ of 76,416 Nmm and 115,536 Nmm occur at a depth of 260 mm and 460 mm, respectively. At these depths of the $M_{\text{max}}$ are 200 mm apart, which is larger than the spacing (100 mm) between two strain gauges. One possible reason for this difference in the $M_{\text{max}}$ locations could have been that, with a large soil movement ($w_s > 60$ mm), the pile tends to move upward from the pulling force (shear stress at soil-pile interface) in response to the dilating soil that is moving upward. Interestingly, the sand dilated approximately 25 mm from the sand surface for all the tests on dense sand ($D_r = 0.89$). Therefore, it appears that the axial load provides a counter reaction to the pulling force from the dilating soil on the pile. Consequently, this counter reaction provides stability for the pile, restraining its movement upward, and causing the $M_{\text{max}}$ to occur at a lower depth, when compared to the pile without an axial load.

As shown in Figure 4.31(b), the $M_{\text{max}}$ is higher in the test with $Q = 294$ N for the whole range of $w_s$. However, only for the test with $Q = 0$ N, $M_{\text{max}}$ reaches its peak (at $w_s = 120$ mm). This trend is consistent with the tests using a 32 mm diameter pile (see Figure 4.30(b)). In summary, the $M_{\text{max}}$ increases with the axial load for both the 32 mm and the 50 mm diameter piles.

![Figure 4.31 Responses of 50 mm diameter pile subjected to different axial loads and arc soil movement profile (0 N and 289 N)](image-url)
4.5 Limiting pressure obtained from model tests

The maximum soil reaction \( r_{\text{max}} \) was obtained for all the model tests, with it being converted to its maximum pressure, \( p_{\text{max}} \) \( (r_{\text{max}}/d \) where \( d \) is the pile diameter). A comparison of the existing limiting pressure proposed for laterally loaded piles is provided by Figure 4.32, which shows the plot of \( p_{\text{max}} \), together with the limiting pressure profiles proposed by Broms (1964) and Barton (1982), respectively. For the ease of comparison, the absolute value of \( p_{\text{max}} \) is taken (generally from the soil reaction profile, \( r_{\text{max}} \) is positive in \( L_m \) and negative in \( L_s \)). It can be seen that \( p_{\text{max}} \) obtained for the majority of the tests are below the limiting pressure proposed by: 1) Broms (1964) for \( L_m/L_s = 200/500 \) (Figure 4.32(a)); and by Barton (1982) for \( L_m/L_s = 400/300 \) (Figure 4.32(b)).

Where \( p_{\text{max}} \) exceeds the limiting pressure by Barton (1982), the tests were conducted with the triangular and the arc soil movement profiles. On these profiles, as \( w_s \) increases, the relative soil movements between the laminar frames of the shear box are generated. In turn, these movements cause shearing between the sand layers, resulting in a more frequent the dilation of the sand, when compared to the test with the rectangular soil movement profile.
Figure 4.32 Comparison of $p_{\text{max}}$ against limiting pressure profiles for laterally loaded piles
4.6 Comparison of pile responses obtained from different soil movement profiles

Tests were undertaken to compare the pile responses obtain from different soil movement profile. The following sections present an overview of the tests.

4.6.1 Tests with $L_m/L_s = 400/300$

The tests conducted on three different soil movement profiles (rectangular, triangular and arc) are shown in Figure 4.33. These tests were conducted with the 50 mm diameter pile on the $L_m$ and the $L_s$ of 400 mm and 300 mm, respectively. It can be seen, from the pile subjected to rectangular soil movement, that the shape of the bending moment profile has double curvature, with both negative and positive values within the $L_m$ and the $L_s$ layers, respectively. This result indicates that some restraints are developed to prevent the pile from rotating, or translating, as freely as, those piles subjected to the triangular and the arc soil movements. Among all the tests, the lowest $M_{\text{max}}$ is recorded for the test with the triangular soil movement profile.

For the test on the pile subjected to an arc soil movement, the bending moment is negative at the sand surface. This value could possibly due to:

1) the initial non-zero reading of the strain gauges before the application of the soil movement (the strain gauge reading not accurately reset to zero prior to the testing); or

2) some restraining effect from the arc soil movement at the surface. The latter appears to be less likely from the data collected from other similar tests (arc).

Figure 4.33(b) shows the deflection profile of the pile subjected to the different soil movement profiles, obtained at the $w_s = 60$ mm (for the tests with the arc and the triangular soil movements, $w_s$ is the soil movement at the sand surface). The deflection profile for the pile subjected to the rectangular soil movement exceeds the magnitude of the soil movement ($w_s = 60$ mm). The pile deflection at the sand surface is 79.69 mm; at approximately 160 mm depth, the pile deflection is equal to the magnitude of the soil movement. As a result, the soil reaction at a depth of up to 100 mm, acting in the opposite direction of the soil movements which also indicates that the soil is restraining the pile from moving forward (see Figure 4.33(c)). In contrast,
for the piles subjected to triangular and arc soil movements, respectively, have pile deflections, for their entire length, that did not exceed the magnitude of the soil movement. Therefore, only the negative soil reaction (in the same direction as the soil movement) is acting on the section of the pile within \( L_m \).

It can be seen from Figure 4.33(d), except for the test with the rectangular soil movement profile, the \( M_{\text{max}} \) increase with \( w_s \), without reaching a peak value.

Figure 4.33(e) shows \( M_{\text{max}} \) plotted against \( S_{\text{max}} \), where \( M_{\text{max}} \) is the absolute value of the maximum bending moment and \( S_{\text{max}} \) is the absolute value of the maximum shear force. It can be seen that \( M_{\text{max}} \) increases linearly with \( S_{\text{max}} \); and that \( M_{\text{max}} \) and \( S_{\text{max}} \) can be fitted in the relation \( M_{\text{max}} = \alpha S_{\text{max}} \), where \( \alpha \) is the coefficient of the linear fit. The lowest and the highest \( \alpha \) are 0.13 (rectangular) and 0.27 (triangular), respectively. These \( \alpha \) values fall within the range of the expression proposed by Guo and Qin (2009), where \( \alpha = 0.103 \) to 0.28 (calculated from Guo and Qin's expression: \( M_{\text{max}} = (0.148 \sim 0.4)PL \), where, \( P = S_{\text{max}} \) and \( L = 0.7 \, \text{m} \) in this study).

The low coefficient of \( \alpha = 0.13 \) obtained from the tests with the rectangular profile, could be attributed to the differences shown in the pile deflection profile (Figure 4.33(b)) for the test with the rectangular profile, when compares to the remaining tests (as discussed above). As a result of these differences, the negative soil reaction and the negative bending moment are generated in \( L_m \), which, in turn, reduces the \( M_{\text{max}} \).
Figure 4.33 Responses of 50 mm diameter pile subjected to three soil movement profiles (L_m = 400 mm, L_s = 300 mm): a) Bending moment profile; b) Pile deflection profile; and c) Soil reaction profile.
Figure 4.33 Responses of 50 mm diameter pile subjected to three soil movement profiles (L_m = 400 mm, L_s = 300 mm): d) $M_{\text{max}}$ against $w_s$; and e) $M_{\text{max}}$ against $S_{\text{max}}$. 

- M_{\text{max}} = 0.13S_{\text{max}} (Rectangular) 
- M_{\text{max}} = 0.22S_{\text{max}} (Arc) 
- M_{\text{max}} = 0.27S_{\text{max}} (Triangular)
4.6.2 Tests with $L_m/L_s = 200/500$

Figure 4.34 shows the comparison of the pile response to different soil movement profiles. The $L_m$ and the $L_s$ are 200 mm and 500 mm, respectively. The six noticeable differences between the responses of the piles are:

1) the bending moment profiles show the shape of a single curvature (Figure 4.34(a))
2) the $M_{\text{max}}$ for the test with the triangular soil movement profile is the highest among all the tests
3) for all three tests, as shown in Figure 4.34(b), the magnitude of the soil movement ($w_s = 120$ mm) exceeds the pile deflection (entire pile length)
4) the magnitude of the soil movement is much larger than the pile deflection, which indicates that the soil flows around the pile, thus, imposing a constant pressure on the pile
5) from Figure 4.34(c), it seem that the soil reaction, resulting from the different soil movement profiles are generally of an arc shape and, is distributed along the depth of the moving layer, $L_m$
6) the difference in $M_{\text{max}}$ at any given $w_s$ is significant between the soil movement profiles. In contrast to Figure 4.33(d) with $L_m/L_s = 400/500$, the $M_{\text{max}}$ for the test with the rectangular and the triangular soil movement profiles show that the $M_{\text{max}}$ reach its peak at $w_s$.

Figure 4.34(e) shows the $M_{\text{max}}$ plotted against $S_{\text{max}}$. It can be seen that for all the soil movement profiles, $M_{\text{max}}$ increases linearly with $S_{\text{max}}$ (with very similar increment rate). Importantly, regardless of the soil movement profile, all the tests results fitted well in the $M_{\text{max}} = \alpha S_{\text{max}}$ relationship, with $\alpha$ ranging between 0.26 and 0.28. This range is consistent with the expression by Guo and Qin (2009).
Figure 4.34 Responses of 32 mm diameter pile subjected to three soil movement profiles (L_m = 200 mm, L_s = 500 mm): a) Bending moment profile; b) Pile deflection profile; and c) Soil reaction profile.
Figure 4.34 Responses of 32 mm diameter pile subjected to three soil movement profiles ($L_m = 200$ mm, $L_s = 500$ mm): d) $M_{\text{max}}$ against $w_s$; and e) $M_{\text{max}}$ against $S_{\text{max}}$.
4.6.3 On general $M_{\text{max}}$ against $S_{\text{max}}$ relationship

A unique relationship was found on the $M_{\text{max}}$ against $S_{\text{max}}$ relationship for the all the tests presented in this chapter (see Tables 4.1, 4.3 and 4.4). Figure 4.35 shows the $M_{\text{max}}$ against $S_{\text{max}}$ relationship for the test results having the following parameters ranging from:

1) soil movement profiles of rectangular, triangular and arc;
2) $L_{\text{ny}}/L_{\text{y}}$ of 200/500 and 400/300;
3) pile diameters of 25 mm, 32mm and 50 mm
4) $Q$ from 0 N to 589 N.

In the $M_{\text{max}} = \alpha S_{\text{max}}$ relationship, $\alpha$ obtained from the linear fit, is 0.24, with a coefficient of uniformity, $r_d^2$, of 0.85. The $\alpha$ obtained is consistent with the $\alpha$ proposed by Guo and Qin (2009), namely, 0.103 to 0.28 (obtained from Guo and Qin's expression: $M_{\text{max}} = (0.148\sim0.4)PL$, where, $P = S_{\text{max}}$ and $L = 0.7$ m for this study).

![Figure 4.35 M\text{max} against S\text{max} relationship for single piles tests (rectangular, triangular and arc soil movement profiles)](image-url)
4.7 Limitations and possible improvements on model tests

The following is a brief overview of the limitations and possible improvements for the model tests.

The distance between where the pile was placed with respect to the shear box boundary, parallel to the direction of the soil movement, had an effect on the pile behaviour. To minimise this effect the pile was placed at the centre of the shear box. Alternatively, a larger shear box could be used to increase the distance between the pile and the shear box boundary.

Further, additional strain gauges (four number of active strain gauges spaced evenly around the circumference of the pile) could have been used to measure the distribution of axial load within the pile, with a full bridge connection. The distribution of axial load can be measured with these strain gauge connections. The existing connection, with two adjacent strain gauges (at each section of the pile), made measuring the axial load on the pile difficult due to the small magnitude of strain; thus it was hard to differentiate the axial strain from the bending strain (as the axial strain is much smaller than the bending strain).

The initial density of the sand bed was successfully controlled and reproduced for each test with the raining technique. During the test, the density of the sand changed immediately with the application of the soil movement (dense sand dilates and loose sand becomes denser). These changes prevented the accurate back calculation and comparison of the limiting pressure profile on the pile (currently the limiting force is compared with the use of the initial density).

The soil movement ($w_s$) was applied by moving the laminar frames forward (as previously mentioned in Chapter 3). The $w_s$ was measured by the amount of the movement of these laminar frames. The actual movement of the sand, within the laminar frames may be different, and may not resemble the movement of the laminar frames. Some indication of the sand movement within the shear box is presented in Subsection 3.2.2. Thus, it would beneficial if a comprehensive investigation assessed
the way sand particles move within the shear box. The resulting findings would help to further interpret the test results.

The application of the axial load onto the pile needed to be carefully carried out, of there would have been p-delta effect onto the pile, even before the soil movement was applied. In order to eliminate this initial p-delta effect, careful measurements were taken to ensure that the weight blocks were placed so that the center of gravity coincided with the pile axis. Also by using weight blocks, there was a limitation on the maximum load that could be applied onto the pile. These limitations can be overcome by using hydraulic jacks for the load application.

4.8 Conclusions

A number of model tests had been conducted on the single piles. A selected number of tests were also repeated to confirm the repeatability of the tests. From these tests, the following general trends in pile behaviour were observed for three soil movement profiles (rectangular, arc and triangular) with the effect from the following parameters: 1) pile diameter; 2) density of sands; 3) axial load; and 4) \( L_m/L_s \) ratio.

*Rectangular soil movement profile*

In relation to the rectangular soil movement profile, at \( L_m/L_s \) ratio of 200/500, it was noted that the pile responded as follows:

- the \( M_{\text{max}} \) and the maximum pile deflection increased proportionally to the relative density of the sand
- the bending moment increased with the diameter of the pile. At above the \( w_s \) of 20 ~ 30 mm, the \( M_{\text{max}} \) on all pile diameters showed little or no increase
- the maximum pile deflection (at the sand surface) reduced as the pile diameter increased
- the shape of the bending moment profile was a single curvature
- the bending moment increased with the axial load.
At the $L_m/L_s$ ratio of 400/300, the pile responded as follows:

- the bending moment and pile deflection increased with the sand density
- as sand density increased from the relative density of 0.27 (loose) to 0.89 (dense), the shape of the bending moment profile changed from a single curvature to double curvature
- the bending moment increased with the pile diameter
- the shape of the bending moment profile was double curvature on the tests without axial load
- as axial load increased, the shape of the bending moment profile changed. $M_{\text{max}}$ reduced while $M_{+\text{max}}$ increased with the increase in the axial load
- the $M_{\text{max}}$ and the maximum pile deflection increased as $L_m/L_s$ increased from 200/500 to 400/300.
- the maximum soil reactions for all tests did not exceed the limiting pressure proposed by Barton (1982).

**Triangular soil movement profile**

At the $L_m/L_s$ ratio of 200/500, the pile responded as follows:

- the bending moment increased with the diameter of pile. At above $w_s = 60$ mm, the $M_{\text{max}}$ on all pile diameter showed little or no increase
- the shape of the bending moment profile was a single curvature
- the bending moment increased with the axial load.

At the $L_m/L_s$ ratio of 400/300, the pile responded as follows:

- the bending moment and the maximum pile deflection increased with the pile diameter
- the shape of the bending moment profile was a single curvature
- the bending moment increased with the axial load.
Arc soil movement profile

At the $L_m/L_s$ ratio of 200/500, the pile responded as follows:

- the bending moment increased with the diameter of the pile. At above $w_s = 60$ mm, the $M_{max}$ on all pile diameter showed little or no increase
- the shape of the bending moment profile was a single curvature
- the bending moment increased with the axial load.

At the $L_m/L_s$ ratio of 400/300, the pile responded as follows:

- the bending moment and the maximum pile deflection increased with the pile diameter
- the shape of the bending moment profile was a single curvature
- the bending moment increased with the axial load.

Comparison among different soil movement profiles

A comparison of the pile responses obtained from the three soil movement profiles indicates that:

- at $L_m/L_s = 200/500$, the shape of the bending moment profiles was a single curvature
- at $L_m/L_s = 400/300$, the shape of the bending moment profile was a double curvature for the test with the rectangular soil movement; and single curvatures for the test with triangular and arc soil movements. Further, the maximum pile deflection, with the rectangular soil movement, exceeded $w_s$, which contributed to the negative soil reaction on the pile in $L_m$. This, in turn, caused the double curvature shape on bending moment profile

An unique linear relationship $M_{max} = \alpha S_{max}$, with $\alpha$ of 0.24, was found by fitting all the tests results presented in this chapter. This relationship was consistent with that proposed by Guo and Qin (2009).
5.0 Introduction

In many cases piles often work in groups, interconnected together with a pile cap, to achieve the desired capacity needed to resist the design load (accumulated load from the column of the superstructure). Piles can be arranged in many different ways; in this chapter, the focus is on studying the piles in groups of two and four piles. These groups of piles were driven into the sand, and restrained with their rigid pile cap located 500 mm above the sand surface. The effects, of a number of parameters on the lateral response of the individual piles within a pile group, were investigated experimentally; the piles were subjected to a rectangular profile of soil movements. Figure 5.1 shows the flowchart outlining the structure of presentation of the pile group tests.
Figure 5.1 Flowchart showing the presentation of the pile group tests
5.1 Tests for pile groups subjected to rectangular soil movement profile

A similar numbering system, used to identify the single pile tests (Chapter 4), is used here to identify the pile group tests. Three additional numbers and letters are added to the system: 1) pile group arrangement; 2) pile group spacing; and 3) location of pile. Figure 5.2 shows a typical test number and the representation of the parameters used in the test.

![Figure 5.2 Test identification number for individual pile within a pile group](image)

**Note:**

* the applied load (Q) is calculated as, mass on the pile (kg) × g (9.81ms⁻²), for example: 60kg × 9.81ms⁻² = 589 N. Also to simplify the identification number kg is presented as k.

All tests presented here are conducted in dense sand at a 600 mm falling height of sand, thus giving \( \gamma = 16.27 \text{ kN/m}^3 \), and a relative density of 0.89. The other parameters varied, namely: 1) the moving (L_m) and the stable (L_s) layers; 2) the pile diameter (d); 3) the axial load applied on the top of the pile cap (Q); 4) the pile group arrangement; and 5) the pile group spacing. These parameters are varied at two different levels, for example, two L_m/L_s ratios of 200/500 and 400/300 respectively, are tested in each pile group type. Table 5.1 shows the tests conducted on piles in a group.
### Table 5.1 Tests conducted on piles in a group

<table>
<thead>
<tr>
<th>Pile group arrangement</th>
<th>Moving / stable layer ((L_m/L_s))</th>
<th>Pile dia. (mm)</th>
<th>Axial load (N)</th>
<th>Pile spacing</th>
<th>Pile Test identification number</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 × 2 (in row)</td>
<td>200/500</td>
<td>32</td>
<td>0</td>
<td>3d</td>
<td>A 2-5 32 0k 600 1x2 3d A_261104</td>
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<tr>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>589</td>
<td>A 4-3 32 60k 600 1x2 3d A_290405</td>
</tr>
<tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>B 4-3 32 60k 600 1x2 3d B_290405</td>
</tr>
<tr>
<td></td>
<td>400/300</td>
<td>32</td>
<td>0</td>
<td>3d</td>
<td>A 4-3 32 0k 600 1x2 3d A_150205</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>589</td>
<td>B 4-3 32 0k 600 1x2 3d B_150205</td>
</tr>
<tr>
<td>2 × 1 (in line)</td>
<td>200/500</td>
<td>32</td>
<td>0</td>
<td>3d</td>
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<td>589</td>
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</tr>
<tr>
<td></td>
<td>400/300</td>
<td>32</td>
<td>0</td>
<td>3d</td>
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</tr>
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<td></td>
<td></td>
<td>589</td>
<td>B 4-3 32 60k 600 2x1 3d B_251005</td>
</tr>
<tr>
<td>2 × 2 (in square)</td>
<td>200/500</td>
<td>32</td>
<td>0</td>
<td>3d</td>
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<td>B 2-5 50 120k 600 2x2 3d B_191104</td>
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<tr>
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<td></td>
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<td>B 2-5 50 120k 600 2x2 3d B_241104</td>
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<tr>
<td></td>
<td>400/300</td>
<td>32</td>
<td>0</td>
<td>3d</td>
<td>A 4-3 32 0k 600 2x2 3d A_220205</td>
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<td>1,177</td>
<td>B 4-3 32 0k 600 2x2 3d B_220205</td>
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<td>B 4-3 32 120k 600 2x2 3d B_161104</td>
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<td></td>
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<td>0</td>
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<td>B 4-3 32 0k 600 2x2 5d B_020305</td>
</tr>
</tbody>
</table>

A large amount of test data is available from the study, for the ease of presentation, the results obtained from each test (Table 5.1) are tabulated in Appendix C. The pile group response is presented in terms of bending moment, shear force, soil reaction, pile rotation and pile deflection for two selected piles (piles A and B). A comparison is made among tests, where mainly the bending moment, the soil reaction and the pile deflection profiles will be presented and discussed in the following sections.
5.2 Setup of pile group testing

The setup of the pile group testing involved a number of adjustments from the standard single pile testing present in Chapter 3. These adjustments are presented as follows:

1) Pile location and instrument locations (Subsection 5.2.1)
2) Pile spacings and pile cap details (Subsection 5.2.2)
3) Pile installation and axial load application (Subsection 5.2.3)
4) Sign convention and interpretation of test results (Subsection 5.2.4)
5) Boundary conditions and limitation of shear box (Subsection 5.2.5).

5.2.1 Pile arrangement and instrument locations

In each pile group test, the response of two instrumented piles in a pile group was recorded. Figure 5.3(a) shows the arrangement of the piles in three different pile groups; the instrumented piles are marked with “A” and “B”. These pile arrangements are: (1) piles in a row (the centreline of the least dimension of the pile cap being in the perpendicular to the direction of the soil movement); (2) piles in a line (the centreline of the least dimension of the pile cap being parallel to the direction of the soil movement); and (3) piles in a square. Due to the limited channels available on the current data acquisition system, the maximum number of strain channels in each test was twenty-four, with one channel being used for temperature compensation (not attached to pile). In addition to the piles used in the single pile tests, two new piles (with 32 mm and 50 mm diameters) were installed with the strain gauges. The positions of the strain gauges along each instrument pile are shown in Figure 5.3(b). The first pile (A) had nine strain gauges; and the second pile (B) had nine strain gauges on the front side and five strain gauges on the opposite side. Two Linear Variable Displacement Transducers (LVDT) were used to measure the rotation and displacement of the pile cap (piles at pile cap level have the same rotation and displacement values); a L-channel plate was used to extend the pile cap’s vertical dimension to achieve a better average on the rotation angle measurement. A future improvement, where more than two instrumented piles are required for a large pile group test, would be replacing the LVDT channels with additional strain gauge
channels. The LVDTs, in turn, can be replaced with two mechanical dial gauges (with an accuracy of ±0.01 mm).

Figure 5.3 Location of piles in a group and strain gauges position

5.2.2 Pile spacings and pile cap details

The pile spacing was one of the parameters known to influence the response of the piles in a group. The two selected pile spacings to be tested were: three times (3d, \(d=\)diameter) and five times (5d) the diameter of the pile, respectively. These spacings (3d and 5d) were measured from the centreline of the pile to the centreline of the nearest adjacent piles. The pile caps were fabricated from a solid aluminium block of 50 mm thick, as shown in Figure 5.4. On one face of the aluminium block, there were predrilled holes of 25 mm deep, with a diameter to fit the piles. The diameter of the holes was made 0.5 mm larger than the pile diameter to facilitate the piles to the pile cap connection. The 0.5 mm allowance made on the predrilled holes enabled the piles (usually with small variation in diameter) to fit into the predrilled holes. The pile fit was reasonably tight on the predrilled holes, with a minimum of movement (\(\leq 0.5\) mm).
mm). Figure 5.4(c) shows that the piles rotate as soil movement is being applied; the maximum angle of rotation of each individual pile is approximately 1°. It should be noted that this small angle of rotation is insignificant and can be easily overcome by the slight application of soil movement. Furthermore, the fixity between the pile and the pile cap is further secured with the application of axial load on the pile cap.

![Figure 5.4 Pile caps details](image)

(a) Pile cap used for piles in a row and piles in a line

(b) Pile cap used for piles in a square

(c) Possible movement between pile and pile cap connection

Note:
1) all dimensions in mm
2) d = the diameter of the pile

Figure 5.4 Pile caps details
5.2.3 Pile installation and axial load application

The installation of the piles in a pile group test was similar to the single pile test described in Chapter 3. The piles in a group were first secured onto the pile cap. The vertical alignment and the spacing of each pile were maintained using a timber spacer (the piles were secured at the middle length of the pile group) during the driving of the pile group into the shear box. The timber spacer, 20 mm thick, was fabricated with predrilled holes having the same spacing as those made on the respective pile cap. The spacer was light weight and could be easily removed once the pile groups had been driven to approximately half their length into the soil. Once the spacer was removed, the remaining length of the pile group could be driven with restraint, provided by the pile cap. A spirit level was used to check the verticality of the pile group; adjustments were made, from time to time, during the driving process to keep the piles in the vertical position. An axial load was applied, using 10 kg weights (or 9.8 N each). For the pile group tests, the weights were placed on the pile cap, and were only secured by the friction between the faces of the pile cap and the weight. This was in contrast to the single pile tests, where a connector was used to secure the weights to the pile head.

5.2.4 Sign convention and interpretation of results

The piles in a group were analysed as individual piles, hence the sign convention used was similar to that described in the single pile tests (Section 4.1). Since all the tests were conducted with a pile cap to secure the individual piles in a group, it was assumed that, at the pile cap level, the pile rotation and the pile deflection were the same for all the piles in a group. This assumption was only valid when all piles in a group remained secured to the pile cap during the entire test.

5.2.5 Boundary conditions and limitation of shear box

The area occupied by a group of piles was significantly greater than that of the single pile tests. Hence, the distance between the face of the pile to the nearest boundary of the shear box was shorter than that of the single pile tests. Figure 5.5 shows the typical location of a pile group in the shear box. Section 4.2.1 (boundary effect on model test) shows the clear distance between the face of the load application (the
boundary where the hydraulic jack was in contact) had a significant effect to the response on the pile. In order to ensure that a consistent comparison was made between the single pile response and the pile group response, a clear distance of 500 mm was maintained from the boundary of the shear box to the centreline of the nearest pile. In contrast, the other side of the pile group had a reduced distance (a minimum of 5d, and varied, depending on the pile group size) from the centreline of the nearest pile to the shear box wall. At the lower section of the pile (in $L_s$), the pile stability depended on the soil reaction, which was similar to the condition of a laterally loaded pile. The current study showed that the lateral soil displacement, around a laterally loaded pile in sand, decreased to less than 5% of the lateral pile displacement, at a distance of 5d (after Tsuchiya et al, 2001). Therefore, with a minimum distance of 5d (>250 mm) for the pile group tests, the boundary effects on the present study were assumed to be insignificant.

Figure 5.5 Location of a pile group in the shear box
Chapter 5: Pile group tests

5.3 Results for piles in a row (1 × 2)

The following Subsections 5.3.1 and 5.3.2 highlight the effect of the axial load on the response of the piles in a row (1 × 2) with \( L_m/L_s \) of 200/500 and 400/300, respectively. All the tests were conducted on the 32 diameter piles at 3d pile spacing.

5.3.1 The effect of axial load with \( L_m/L_s \) of 200/500

The effect of the axial load on the 1 × 2 pile group with \( L_m/L_s = 200/500 \) was investigated by conducting two tests: one test without the axial load (Q = 0 N); and another test with the axial load, applied on top of the pile cap (Q = 589 N). Figures 5.6 (a) and (b) present the bending moment profiles obtained from the instrumented piles A and B. Since this pile group is geometrically symmetrical, with respect to the direction of the soil movement (see Figure 5.3(a)), the responses of piles A and B are expected to be similar. This similarity in the pile response can be seen for both tests, as the difference in the maximum positive bending moments (\( M_{+\text{max}} \)) between piles A and B is in the order of 16%. As shown in Figures 5.6 (c) to (f), the \( M_{\text{max}} \) and \( r_{\text{max}} \) (note that \( M_{\text{max}} \) refers to both \( M_{+\text{max}} \) and \( M_{-\text{max}} \); similarly, \( r_{\text{max}} \) refers to both \( r_{+\text{max}} \) and \( r_{-\text{max}} \)) increase with the \( w_s \) up to 30 mm. Further increase in the \( w_s (>30 \text{ mm}) \), both the \( M_{\text{max}} \) and \( r_{\text{max}} \), remain almost constant.

Importantly, the lower value of the bending moment obtained for the test with Q = 589 N is resulted from an error which occurred during the experiment. While applying the lateral soil movement, the connection between the hydraulic jack and the loading block bent due to a high eccentric loading, created when the axis of the hydraulic jack did not coincide with the centre of the passive pressure developed in the sand (Figure 5.7).

In the subsequent tests, the problem was rectified with a new hydraulic jack. The Enerpac RC-59 (Enerpac, 2002), with a saddle diameter of 25 mm (where the loading block was connected to the hydraulic jack, see Figure 5.6) was replaced by the new larger capacity hydraulic jack, the Enerpac RC-108 with a saddle diameter of 35 mm. The aims of this modification were:

1) to provide a bigger section at the highly concentrated stress point
2) to minimise the eccentric loading by ensuring that the centre of the loading block coincided with the axis of the hydraulic jack.

In summary, the effect of the axial load on the 1 × 2 pile group at $L_m/L_s = 200/500$ could not be identified due to the abovementioned experimental problem. However, as indicated by the results, the responses of piles A and B, respectively, are similar in each test.
Figure 5.6 Response of 32 mm diameter piles in a row (L_s/L_m = 200/500)

(a) Pile A, w_s=60 mm

(b) Pile B, w_s=60 mm

(c) M_max and M_min against w_s (Pile A)

(d) M_max and M_min against w_s (Pile B)

(e) r_max and r_min against w_s (Pile A)

(f) r_max and r_min against w_s (Pile B)
Figure 5.7 Bending failure on the connection between the hydraulic jack and the rectangular loading block
5.3.2 The effect of axial load with $L_m/L_s$ of 400/300

The effect of the axial load on the $1 \times 2$ pile group with $L_m/L_s = 400/300$ was investigated by conducting two tests with $Q = 0$ N and 589 N, respectively. The bending moment profiles of the piles are shown in Figures 5.8(a) and (b) at $w_s = 60$ mm. The difference in the $M_{r\text{max}}$ between piles A and B, with $Q = 0$ N, is 9.3 %. This difference increases to 58.6 % with $Q = 589$ N. On piles A, as Q increases from 0 N to 589 N, the $M_{r\text{max}}$ increases by 27.3 %. In contrast, on piles B, as Q increases from 0 N to 589 N, the $M_{r\text{max}}$ reduces by 14%. As the soil movement increases to $w_s = 120$ mm, the $M_{r\text{max}}$ on both piles A and B, with $Q = 589$ N are higher than that with $Q = 0$ N (Figures 5.8(c) and (e)).

One contribution to the differences in the bending moment response between piles A and B could be due to the following experimental shortcomings:

1) the asymmetrical distribution of the stress from the within the shear box when the soil moves, which leads to a torsion moment developing on the pile group (the current instrumentation did not allow for this to be measured)

2) the installation process of the pile group, which could lead to a less than prefect symmetrical position of the pile group, could result in an uneven distribution of the soil reaction onto the pile.

3) the application of Q, if not at the centre of the pile cap, could create an eccentric moment on the piles

4) the pile group, if not perfectly vertical after being driven into the sand, could cause a p-delta effect when Q is applied (even before $w_s$ is applied).

Figures 5.8(e) and (h) shows that the $M_{r\text{max}}$ and $r_{r\text{max}}$ increase with $w_s$, however no definite peak has been noted. These increments also show that the $M_{r\text{max}}$ and $r_{r\text{max}}$ increase at different rates: initially slower at $w_s < 60$ mm; and subsequently higher at $w_s > 60$ mm. This result is in contrast to the tests with $L_m/L_s = 200/500$ (Subsection 5.3.1), where both the $M_{r\text{max}}$ and $r_{r\text{max}}$ reach their peak at $w_s = 30$ mm. In summary, it can be seen that the $M_{r\text{max}}$ increases with Q, more noticeably at a higher $w_s (= 120$ mm).
Figure 5.8 Response of 32 mm diameter piles in a line (Lₙ/Lₘ = 400/300)
Chapter 5: Pile group tests

Figure 5.8 Response of 32 mm diameter piles in a line ($L_s/L_m = 400/300$)

(g) $r_{\max}$ and $r_{\max}$ against $w_s$ (Pile A)

(h) $r_{\max}$ and $r_{\max}$ against $w_s$ (Pile B)
Chapter 5: Pile group tests

5.4 Results of piles in a line (2 × 1)

The following Subsections 5.4.1 and 5.4.2 present an overview of four tests that investigated the effect of the axial load on the response of the piles in line (2 × 1). The tests were conducted with 32 mm diameter piles at a 3d pile spacing.

5.4.1 The effect of axial load with L_m/L_s of 200/500

In order to investigate the effect of the axial load on the 2 × 1 pile group at L_m/L_s = 200/500, two tests were conducted with Q being 0 N and 589 N, respectively. The response, in terms of the bending moment and the soil reaction of the piles, is shown in Figure 5.9. The following pile behaviour are identified:

- for the test Q = 589 N, the M_{max} on pile A, at a depth of 460 mm, is 81% higher the test with Q = 0 N
- it is expected that this pile group arrangement would enable the pile cap to provide additional restraint against the rotation on the pile, which would then lead to a significant negative moment at the surface of the soil (depth = 0 mm). This restraint is noticeable on pile A (with a negative moment at the surface on both tests)
- in contrast to pile A, less restraint is provided on pile B (also known as the shadowing or trailing pile), as can be seen in Figure 5.9(b), indicating a lesser negative moment at the soil surface. Thus it appears that the position of the piles plays a role in the bending moment distribution between the piles in this group.

The axial load distribution between the piles in a pile group may be different, depending on the location of each individual pile. This distribution is not investigated here, as it could not be measured with the current instruments and data acquisition system. The difference in the M_{max} between piles A and B is 99.0 % and 8.6 %, for the tests with Q = 0 N and Q = 589 N, respectively. Pile B has recorded a higher M_{max} in both tests. On both piles A and B, the M_{max} and r_{max} reach their peak at w_s = 40 mm (Figures 5.9(c) to (f)). In summary, the M_{max} on piles A increases with Q, while for piles B, Q has insignificant effect on the bending moment.
Chapter 5: Pile group tests

Figure 5.9 Response of 32 mm diameter piles in a line (L_s/L_m = 200/500)
5.4.2 The effect of axial load with \( L_m/L_s \) of 400/300

Following the above experiment, the effect of the axial load on the response of the 2 \( \times \) 1 pile group was further investigated, with \( L_m/L_s \) being increased to 400/300. The bending moment and the soil reaction responses of piles A and B are shown in Figure 5.10. Unfortunately, while conducting the test with \( Q = 0 \) N, a failure (damage due to bending) occurred at the connection between the loading block and the hydraulic jack. The bending of the loading block prevented the laminar frames from moving forward in a rectangular profile. As can be seen in Figure 5.6, instead of moving forward uniformly, the failure caused the laminar frames to move in a trapezoidal shape, with a maximum movement at the top. This movement profile was different from the expected rectangular profile, and thus caused the pile responses to act differently. A similar failure also occurred on one of the tests reported in Subsection 5.3.1. However, it was only later in the experimental program that the problem was able to be rectified with a larger sized hydraulic jack. Due to this failure on the test with \( Q = 0 \) N, the magnitude of the pile responses should have been lower, while the shape of the profiles remained unchanged.

A comparison of piles A and B shows the differences in the \( M_{\text{max}} \) for tests with \( Q = 0 \) N and \( Q = 589 \) N, namely 21.0 % and 56.6 %, respectively. Piles B has recorded a higher \( M_{\text{max}} \) in both tests. Further, as shown in Figures 5.10(c) to (f), for the test with \( Q = 589 \) N, the \( M_{\text{max}} \) and \( r_{\text{max}} \) reach their peak at \( w_s \sim 60 \) mm. At \( w_s > 60 \) mm, the pile responses generally remain unchanged. The \( M_{\text{max}} \) and \( r_{\text{max}} \) for the test with \( Q = 0 \) N could be effected by the failure mentioned above. In both tests, the depth of the \( M_{\text{max}} \) is at a depth of 460 mm, approximately, for piles A and B, respectively. In summary, the effect of the axial load could not be determined for this pile group due to the experimental problem mentioned above. Nevertheless, it can be established that, on the 2 \( \times \) 1 pile group, the \( M_{\text{max}} \) on pile A is lower than that on pile B.

A further comparison from the previous Subsection 5.4.1, of the tests with \( Q = 589 \) N, shows that the \( M_{\text{max}} \), obtained for piles A and B, on the test with \( L_m/L_s = 400/300 \) are 7 and 10 times those obtained from the test with \( L_m/L_s = 200/500 \), respectively. On the other hand, a comparison of the negative bending moment at the soil surface, for
Chapter 5: Pile group tests

piles A and B, shows that they are 0.6 and 2.7 times that obtained from the tests with \( L_m/L_s = 200/500 \). 
Chapter 5: Pile group tests

Figure 5.10 Response of 32 mm diameter piles in a line (L_s/L_m = 400/300)

(a) Bending moment profile of Pile A
(b) Bending moment profile of Pile B
(c) $M_{+\text{max}}$ and $M_{-\text{max}}$ against $w_s$ (Pile A)
(d) $M_{+\text{max}}$ and $M_{-\text{max}}$ against $w_s$ (Pile B)
(e) $r_{+\text{max}}$ and $r_{-\text{max}}$ against $w_s$ (Pile A)
(f) $r_{+\text{max}}$ and $r_{-\text{max}}$ against $w_s$ (Pile B)

Figure 5.10 Response of 32 mm diameter piles in a line (L_s/L_m = 400/300)
5.5 Results of piles in square arrangement (2 × 2)

The piles in a square arrangement (2 × 2) were studied more extensively than the other pile groups. The following Subsections 5.5.1 and 5.5.2 present an overview of the effect of the axial load on the pile group response. The tests were conducted by adopting the following:

1) pile diameters of 32 mm and 50 mm
2) pile spacings of 3d and 5d
3) $L_m/L_s$ ratios of 200/500 and 400/300.

5.5.1 Test on 32 mm diameter piles with $L_m/L_s$ of 200/500

Four tests were conducted to investigate the effect of the axial load on the 2 × 2 pile group with $L_m/L_s = 200/500$. The following Subsections 5.5.1.1 and 5.5.1.2 provide an overview of the tests conducted on the 32 mm diameter pile at pile spacings of 3d and 5d, respectively.

5.5.1.1 Piles in 3d spacing

The effect of the axial load on the 2 × 2 pile group with 3d pile spacing was investigated using a $Q$ of 0 N and 1,177 N on the tests, respectively. The following information is obtained from the bending moment profiles of piles A and B, shown in Figures 5.11(a) and (b), respectively:

1) the maximum negative and positive bending moments of pile A, on both tests, are at the depth of 60 mm and 460 mm, respectively
2) for pile A, the ratios of $M_{+\text{max}}/M_{-\text{max}}$ are 1.8 and 1.14 for the tests with $Q = 0$ N and 1,177 N, respectively
3) pile B behaved differently with the load $Q$. The bending moment profile of pile B, with $Q = 0$ N, has single curvature shape with only a positive bending moment along the entire length of the pile, and a $M_{+\text{max}}$ of 8,771 Nmm, at a depth of 360 mm. On the other hand, pile B with $Q = 1,177$ N, shows a $M_{+\text{max}}$ of 5,890 Nmm at a depth of 560 mm, while the negative bending moment appears to extend toward the pile cap. The difference in the bending moment profiles indicates that the deflection mode of pile B changes with the axial load. On the pile cap level, the presence of the axial load could have prevented...
the pile cap from rotating in a clockwise direction, which effectively provides additional restraint to the piles.

Figures 5.11(c) to (f) show the increase of the $M_{\text{max}}$ and $r_{\text{max}}$ with $w_s$, for piles A and B. The common features of the $2 \times 2$ pile group tests are presented in Section 5.5.4. In summary, the $M_{+\text{max}}$ and $M_{-\text{max}}$ on pile A increases with $Q$, while the $M_{+\text{max}}$ on pile B reduces as $Q$ increases.
Figure 5.11 Response of 2 × 2 pile group at 3d pile spacing (Ls/Lm = 200/500)
5.5.1.2 Piles in 5d spacing

Two additional tests were conducted with the same parameters as the tests with the 3d pile spacing (Subsections 5.5.1.1), but with the pile spacing being increased to 5d. Figures 5.12(a) and (b) show the bending moment profiles obtained from the two tests with $Q = 0$ N and 1,177 N, respectively. From the figures it can be seen that:

1. the negative bending moments (piles A and B) are located at the soil surface
2. for pile A, on both tests ($Q = 0$ N and 1,177N), the ratios of the $M_{\text{max}}/M_{\text{+max}}$ are between 0.7 and 0.8, which indicate that the $M_{\text{+max}}$ is slightly higher than the $M_{\text{max}}$
3. On the other hand, for pile B, where the $M_{\text{+max}}/M_{\text{max}}$ are 2.33 and 4.55 for the tests with $Q = 0$ N and 1,177 N, respectively, there are significant differences between the $M_{\text{max}}$ and $M_{\text{+max}}$
4. the $M_{\text{+max}}$ for piles A and B developed at the same depths of 460 mm and 360 mm, respectively, regardless of $Q$.

Figure 5.12(c) to (f) show the increase of $M_{\text{max}}$ and $r_{\text{max}}$ with $w_s$, for piles A and B. A number of common features drawn from other $2 \times 2$ pile group are discussed in Subsection 5.5.4. In summary, the axial load as insignificant influence on pile A, but on pile B, the $M_{\text{+max}}$ is significant reduces as $Q$ increases.
Figure 5.12 Response of 2 × 2 pile group at 5d pile spacing (Lₜ/Lₘ = 200/500)
5.5.2 Test on 32 mm diameter pile with $L_m/L_s$ of 400/300

Another set of four tests were conducted to investigate the effect of axial load with the same parameters described in Subsection 5.5.1, but with increased $L_m/L_s$ of 400/300. The following Subsections 5.5.2.1 and 5.5.2.2 present the 2 x 2 pile group tests conducted at different pile spacings of 3d and 5d, respectively.

5.5.2.1 Piles in 3d spacing

The effect of axial load on the 2 x 2 pile group with 3d pile spacing was investigated by two tests conducted with $Q$ of 0 N and 1,177 kN, respectively. The results in term of the bending moment response for piles A and B are shown in Figures 5.13(a) and (b), respectively. The following pile behaviour can be identified from the figures:

1) test with $Q = 0$ N, negative bending moments at the soil surface develop on piles A and B. These moments extend to a depth up to approximately 260 mm
2) as $Q$ increases from 0 N to 1,177 N, the $M_{max}$ for piles A and B increases by 99% and 94%, respectively
3) the depth of the $M_{max}$ is at 460 mm, for both piles in both tests.

Figures 5.13(c) to (f) show the increase in the $M_{max}$ and $r_{max}$ with $w_s$, for piles A and B. The following response to $w_s$ for both piles A and B can be identified:

1) the $M_{max}$ and $r_{max}$ increase with $w_s$ without reaching a definite peak values
2) the $M_{max}$ for the test with $Q = 1,177$ N are consistently higher than that test with $Q = 0$ N by approximately 56%, for the entire range of $w_s$
3) At $w_s = 60$ mm, the $M_{max}$ at the soil surface reduces, and becoming positive bending moment, as $Q$ increases from 0 N to 1,177 N.

In summary, it can be clearly seen that, on both piles A and B, the $M_{max}$ increases with $Q$. 
Figure 5.13 Response of $2 \times 2$ pile group at 3d pile spacing ($L_s/L_m = 400/300$)
5.5.2.2 Piles in 5d spacing

The effect of the axial load on the 2 × 2 pile group was further investigated with two tests conducted with Q = 0 N and 1,177 N, having the same parameters as described in Subsection 5.5.2.1, but with pile spacing increased to 5d. Figures 5.14(a) and (b) presents the bending moment of piles A and B, respectively, obtained at the soil movement of ws = 60 mm. It should be noted that, during the test, pile B (with Q = 0 N) was driven with strain gauges facing in the wrong loading direction (the strain gauges should have faced the direction of the soil movement in order to capture the bending strain on the pile). As a result the bending moments recorded were low, since the bending stresses at the neutral axis of the pile were almost zero. In view of this, the following discussions only focus of pile A of the tests. The following pile behaviour can be identified:

1. The M_{max} and M_{+max} of pile A (Q = 0 N) are -9,950 Nmm (at depth = 260 mm) and 6,537 Nmm (at depth = 560 mm), respectively. These moments give the M_{+max}/M_{max} of 0.66.

2. With the axial load (Q = 1,177 N), the M_{max} and M_{+max} of pile A are -15,619 Nmm (at depth = 160 mm) and 9,111 Nmm (at depth = 560 mm). These moments give the M_{+max}/M_{max} of 0.58.

Figures 5.14 (c) to (f) show the increase of the M_{+max} and r_{max} with ws, for piles A and B. For completeness, the results of pile B (marked “?”) is also included. However, the following pile response with respect to ws is identified for pile A only:

1. The M_{max} and r_{max} increase with ws, and no definite peak values are reached.

2. While the M_{+max} and r_{max} increase in magnitude on both tests (Q = 0 N and 1,177 N), the M_{max} changes from negative to positive, as ws increases beyond 100 mm (p-delta effect begin to dominate).

In summary, from the pile behaviour identified above, it can be seen that the axial load increases both the M_{+max} of pile A.
Figure 5.14 Response of $2 \times 2$ pile group at 5d pile spacing ($L_s/L_m = 400/300$)
5.5.3 Tests on 50 mm diameter piles with \( L_m/L_s \) of 200/500

A series of four tests, using 50 mm diameter piles (4.5 times the stiffness of the 32 mm diameter pile, Subsection 5.5.1) investigated the effect of the axial load on the 2 \( \times \) 2 pile group with \( L_m/L_s = 200/500 \). The following Subsections 5.5.3.1 and 5.5.3.2 present an overview of the tests conducted (with the stiffer piles) at 3d and 5d pile spacings, respectively.

5.5.3.1 Piles in 3d spacing

The effect of the axial load on the 2 \( \times \) 2 pile group with 3d pile spacing was investigated by conducting two tests with \( Q = 0 \) N and 1,177 N, respectively. Figures 5.15(a) and (b) show the bending moment responses of piles A and B, respectively. The following pile behaviour can be identified from these figures:

1) at a depth of 60 mm, the \( M_{\text{max}} \) of pile A are -19,392 Nmm and -31,320 Nmm, for the tests with \( Q = 0 \) N and \( Q = 1,177 \) N, respectively.
2) the entire length of pile A, in the test with \( Q = 1,177 \) N, has negative bending moments.
3) in contrast to \( Q = 1,177 \) N, in the test with \( Q = 0 \) N, pile A has both negative and positive bending moments (\( M_{+\text{max}} \) at depth = 560 mm).
4) the difference in the bending moment between the tests with \( Q = 0 \) N and 1,177 N (on pile A) is consistently between 11,448 Nmm \( \sim \) 18,552 Nmm. This consistency indicates that as \( Q \) increases from 0 N to 1,177 N, a constant negative bending moment is induced from the pile cap level down to the toe of pile A. One possible factor for inducing such bending moment could be the result of an imperfect pile group loading, where the centre of gravity of the weights (axial load) being applied away from the centreline of the pile cap.

Figures 5.14 (c) to (f) show that the \( M_{\text{max}} \) and \( r_{\text{max}} \) increase with \( w_s \), for piles A and B. For ease of presentation, these figures are discussed later in Subsection 5.5.4. The effect of the axial load cannot be summarised from these tests due to the inconsistency in the pile behaviour, a result of the imperfect application of the axial load onto the pile cap.
Figure 5.15 Response of $2 \times 2$ pile group at $3d$ pile spacing ($L_s/L_m = 200/500$)
5.5.3.2 Piles in 5d spacing

The effect of the axial load on the 2 × 2 pile group was further investigated for Q = 0 N and 1,177 N, with the same parameters as described in Subsection 5.5.3.1, but with pile spacing increased to 5d. Figures 5.16(a) and (b) present the bending moment for piles A and B, respectively, and show the following pile behaviour:

1) the M+max for pile A are -14,376 Nmm and 21,696 Nmm for tests with Q = 0 N and Q = 1,177 N, respectively
2) the ratios of the M−max/M+max are 1.71 and 0.66 for tests with 0 N and 1,177 N axial load, respectively
3) the increase in the positive bending moment (pile A), when the axial load is added indicates that the axial load contributes additional moment to the pile (p-delta effect)
4) in contrast to pile A, the p-delta effect is not evident on pile B, where both the bending moment profiles obtained from the tests with Q = 0 N and 1,177 N, respectively, are similar in shape and magnitude.

A comparison made between the results (for Q = 0 N), presented in Figures 5.15(a) and 5.16(a), shows that, by increasing the pile spacing from 3d to 5d, additional restraint against rotation is provided at the pile cap level, leading to a higher M−max on pile A. However, this result contrasts with the result for the test with Q = 1,177 N, where the presence of the axial load forces the pile cap to rotate in a clockwise direction, which effectively reduces the pile cap restraint. As a result, M−max at the pile cap level reduces.

Figures 5.16 (c) to (f) show the increase of the Mmax and rmax with ws, for piles A and B. For ease of presentation, these figures will be discussed later in Subsection 5.5.4. In summary, from the pile behaviour identified above, it can be seen that the axial load increases both the M+max and the M−max of pile A; however it has a negligible impact on pile B.
Figure 5.16 Response of $2 \times 2$ pile group at 5d pile spacing ($L_s/L_m = 200/500$)
5.5.4 Summary of common trends noted on the 2 × 2 pile groups

Table 5.2 Reference to the 2 × 2 pile groups tests

<table>
<thead>
<tr>
<th>$L_m/L_s$ (mm)</th>
<th>Pile diameter (mm)</th>
<th>Pile spacing</th>
<th>Subsection</th>
<th>Figure</th>
</tr>
</thead>
<tbody>
<tr>
<td>200/500</td>
<td>32</td>
<td>3d</td>
<td>5.5.1.1</td>
<td>5.11(c) to (f)</td>
</tr>
<tr>
<td></td>
<td>5d</td>
<td>5d</td>
<td>5.5.1.2</td>
<td>5.12(c) to (f)</td>
</tr>
<tr>
<td>400/300</td>
<td>32</td>
<td>3d</td>
<td>5.5.3.1</td>
<td>5.14(c) to (f)</td>
</tr>
<tr>
<td></td>
<td>5d</td>
<td>5d</td>
<td>5.5.3.2</td>
<td>5.15(c) to (f)</td>
</tr>
</tbody>
</table>

(a) 2 × 2 pile group with $L_m/L_s = 200/500$

A number of similarities are noted on the $M_{max}$ and $r_{max}$ response to an increasing $w_s$ for the test with $L_m/L_s = 200/500$ (see Table 5.2), as follows:

1) on piles A and B, the $M_{max}$ and $r_{max}$ increases with $w_s$ up to $w_s = 40$ mm
2) on piles A and B, at $w_s > 40$ mm, generally, the $M_{max}$ and $r_{max}$ reach their peak and remain unchanged as $w_s$ increases
3) regardless of the pile spacing, the difference in pile response ($M_{max}$ and $r_{max}$) on pile B between the tests with $Q = 0$ N and 1,177 N, respectively, is minor
4) on all tests, the $r_{max}$ on pile A are significantly higher than that obtained from pile B.

(b) 2 × 2 pile group with $L_m/L_s = 400/300$

A summary of the common features for the tests with $L_m/L_s = 400/300$ is listed below, regardless of $Q$, the pile spacing or the pile position (A or B):

1) the $M_{max}$ and $r_{max}$ increase with $w_s$; no definite peak value are reached
2) the $M_{max}$ and $r_{max}$ increase with $Q$. 
5.6 The effect of $L_m/L_s$ ratio

The effect of the $L_m/L_s$ ratio was investigated by comparing the tests, with $L_m/L_s$ of 200/500 and 400/300, presented in the earlier Sections, according to the pile group:

- 1 × 2 pile group, Subsection 5.6.1
- 2 × 1 pile group, Subsection 5.6.2
- 2 × 2 pile group, Subsection 5.6.3.

It should be noted that, all the tests presented in this section used 32 mm diameter piles, 3d pile spacing and $Q = 0$ N.

5.6.1 The effect of $L_m/L_s$ on piles in a row (1 × 2)

The effect of the $L_m/L_s$ ratio on the 1 × 2 pile group was investigated by comparing two tests previously presented in Section 5.3. Table 5.3 shows the comparison between the $M_{\text{max}}$ obtained from the pile group tests. The $M_{\text{max}}$ of pile A from the test with $L_m/L_s = 400/300$ is 2.5 times that of the test with $L_m/L_s = 200/500$, with only a small difference in the $M_{\text{max}}$ (13%) between piles A and B of each test.

Figures 5.17(a) and (b) shows the bending moment response from the tests. For comparison, the pile response was taken at $w_s = 100$ mm instead of $w_s = 120$ mm as the maximum $w_s$ applied on the single pile tests was < 120 mm. The comparison between the single pile test (Chapter 4) and the pile group test shows that:

1) at $L_m/L_s = 200/500$, the bending moment profiles are similar in shape; however, the $M_{\text{max}}$ of the single pile is up to 43% higher than that of piles A and B.

2) at $L_m/L_s = 400/300$, the bending moment profiles are similar in shape; however, the $M_{\text{max}}$ of the single pile is up to 30% higher than that of piles A and B.

3) the bending moments recorded for piles A and B in the pile group ($L_m/L_s = 400/300$), at a depth of 260 mm, are 3,244 Nmm and 4,716 Nmm, respectively, compare to -8,200 Nmm recorded for the single pile test.

4) the bending moment profiles of the pile group are mainly positive bending moment (the small negative bending moment on pile A could have been due to the asymmetrical loading on the pile group, Subsection 5.3.2), while the
bending moment profile of the single pile has a significant negative and positive bending moments, which resulted from a double curvature shape.

Figure 5.17(c) shows the deflection profiles of the piles. From the profiles, the following pile behaviours are identified:

1) at the surface, the deflection of the single pile at $L_m/L_s = 400/300$ is 150.5 mm, and exceeds the soil movement ($w_s = 100$ mm).
2) generally, the deflection profiles are a single curvature, with the exception of the single pile test at $L_m/L_s = 400/300$, which shows a double curvature. As previously discussed in Chapter 4, when the pile deflection (at the soil surface) is more than the soil movement, a negative soil reaction (acting in the opposite direction of the soil movement) is generated. This negative soil reaction subsequently causes the pile to bend backward (negative bending moment) with respect to the direction of the soil movement. As a result, the bending moment and the deflection profiles have a double curvature shape (due to the scale of the magnitude it is noticeable on the bending moment profile but less on the deflection profile).

Figure 5.17(d) shows the $M_{\text{max}}$ against the soil movement between the single pile and the pile group, indicating that:

1) in the pile group test with $L_m/L_s = 200/500$, the $M_{\text{max}}$ reaches a peak of 26,798 Nmm at $w_s = 30$ mm. Above $w_s > 30$ mm, the $M_{\text{max}}$ remains almost constant
2) in the single pile and the pile group tests with $L_m/L_s = 400/300$, the maximum value of $M_{\text{max}}$ are slowly attained, when compared to the tests with $L_m/L_s = 200/500$
3) in the tests with $L_m/L_s = 400/300$ and $w_s > 80$ mm, the $M_{\text{max}}$ on the single pile test is consistently 30% higher than that of the pile in the pile group test
4) in the tests with $L_m/L_s = 400/300$, the $M_{\text{max}}$ increases with $w_s$, however, this increment is slow above $w_s > 80$ mm.
Figure 5.17 The effect of $L_m/L_s$ ratio on the pile response of $1 \times 2$ pile group

Table 5.3 Summary of the effect of $L_m/L_s$ ratio on the piles in a row ($w_s = 60$ mm)

<table>
<thead>
<tr>
<th>Pile A (coloured in black)</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>Ratio $M_{\text{max}}/M_{\text{max}}$</th>
<th>Pile B (coloured in black)</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>Ratio $M_{\text{max}}/M_{\text{max}}$</th>
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<tbody>
<tr>
<td>$L_m = 200$ mm</td>
<td>27,622</td>
<td>-1,575</td>
<td>17.54</td>
<td>$L_m = 200$ mm</td>
<td>23,841</td>
<td>-599</td>
<td>39.80</td>
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<td>$L_s = 500$ mm</td>
<td></td>
<td></td>
<td></td>
<td>$L_m = 400$ mm</td>
<td>70,933</td>
<td>-2685</td>
<td>26.41</td>
</tr>
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<td>Cycle:</td>
<td></td>
<td></td>
<td></td>
<td>$L_s = 300$ mm</td>
<td>70,749</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>Maximum bending moment (Nmm)</td>
<td></td>
<td></td>
<td></td>
<td>Maximum bending moment (Nmm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth (mm)</td>
<td></td>
<td></td>
<td></td>
<td>Depth (mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pile deflection (mm)</td>
<td></td>
<td></td>
<td></td>
<td>Soil movement, $w_s$ (mm)</td>
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</tbody>
</table>

Maximum bending moment ratio between pile groups ($L_m/L_s = 200/500$ and $400/300$) 0.39 0.59 n/a
5.6.2 The effect of $L_m/L_s$ on piles in a line ($2 \times 1$)

The effect of the $L_m/L_s$ ratio on piles in a line was investigated by comparing two tests having an $L_m/L_s$ of 200/500 and 400/300, respectively (see Section 5.4). The bending moment profiles at $w_s = 60$ mm are shown in Figure 5.18. Generally, the shape of the bending moment profile on piles A and B is similar. As shown in Table 5.4, the ratios of the $M_{max}$, between the tests with $L_m/L_s = 200/500$ and 400/300, are 0.14 and 0.10, for piles A and B, respectively. These ratios, in turn show that the $M_{max}$ increases with the $L_m/L_s$ ratio.

![Figure 5.18](image)

**Figure 5.18** The effect of $L_m/L_s$ on the bending moment profile of 2 $\times$ 1 pile group

**Table 5.4 Summary of the effect of $L_m/L_s$ ratio on the piles in a line ($w_s = 60$ mm)**

<table>
<thead>
<tr>
<th>Pile A (coloured in black)</th>
<th>$M_{max}$ (Nmm)</th>
<th>$M_{max}$ (Nmm)</th>
<th>Ratio $M_{max}/M_{max}$</th>
<th>Pile B (coloured in black)</th>
<th>$M_{max}$ (Nmm)</th>
<th>$M_{max}$ (Nmm)</th>
<th>Ratio $M_{max}/M_{max}$</th>
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</thead>
<tbody>
<tr>
<td>$L_m = 200$ mm, $L_s = 500$ mm</td>
<td>11,213</td>
<td>-11,013</td>
<td>1.02</td>
<td>$L_m = 200$ mm, $L_s = 500$ mm</td>
<td>12,180</td>
<td>-6,933</td>
<td>1.76</td>
</tr>
<tr>
<td>$L_m = 400$ mm, $L_s = 300$ mm</td>
<td>78,197</td>
<td>-5,171</td>
<td>15.12</td>
<td>$L_m = 400$ mm, $L_s = 300$ mm</td>
<td>122,484</td>
<td>-14,386</td>
<td>8.51</td>
</tr>
<tr>
<td>$M_{max}$ ratio between pile groups ($L_m/L_s = 200/500$ and 400/300)</td>
<td>0.14</td>
<td>2.13</td>
<td>n/a</td>
<td>$M_{max}$ ratio between pile groups ($L_m/L_s = 200/500$ and 400/300)</td>
<td>0.10</td>
<td>0.48</td>
<td>n/a</td>
</tr>
</tbody>
</table>
5.6.3 The effect of $L_m/L_s$ on piles in a square (2 × 2)

Figure 5.19 presents the bending moment profiles of piles in a square arrangement, tested with $L_m/L_s = 200/500$ and $400/300$, respectively. The bending moment profiles of the piles have a double curvature shape, with the exception of pile B in the test with $L_m/L_s = 200/500$ (see discussion in Subsection 5.5.1.1). The soil reaction profiles (Appendix C) indicate that there is no negative soil reaction acting on the piles within $L_m$. Therefore, the negative bending moments recorded for the piles (A and B) are the result of the restraint provided by the pile cap and the interaction between the adjacent piles. Table 5.5 summarises the different ratios of the $M_{+\text{max}}$ and $M_{-\text{max}}$. On each pile, the $M_{+\text{max}}$ is significantly higher than the $M_{-\text{max}}$. The ratio of the $M_{+\text{max}}$ between the pile groups with $L_m/L_s = 200/500$ and $400/300$ is 0.31 and 0.25, for piles A and B, respectively. These ratios show that the $M_{+\text{max}}$ (for both piles A and B) increases with $L_m/L_s$. 

---

173
Chapter 5: Pile group tests

Figure 5.19 The effect of $L_m/L_s$ on the bending moment profile of $2 \times 2$ pile group

Table 5.5 Summary of the effect of $L_m/L_s$ ratio on the piles in a square ($w_s = 60$ mm)

<table>
<thead>
<tr>
<th>Pile A (coloured in black)</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>Ratio $M_{\text{max}}/M_{\text{max}}$</th>
<th>Pile B (coloured in black)</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>Ratio $M_{\text{max}}/M_{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_m = 200$ mm, $L_s = 500$ mm</td>
<td>9,943</td>
<td>-5,539</td>
<td>1.80</td>
<td>$L_m = 200$ mm, $L_s = 500$ mm</td>
<td>8,712</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>$L_m = 400$ mm, $L_s = 300$ mm</td>
<td>31,921</td>
<td>-4,420</td>
<td>7.22</td>
<td>$L_m = 400$ mm, $L_s = 300$ mm</td>
<td>35,055</td>
<td>-2,661</td>
<td>13.17</td>
</tr>
<tr>
<td>$M_{\text{max}}$ ratio between pile groups ($L_m/L_s = 200/500$ and $400/300$)</td>
<td>0.31</td>
<td>1.25</td>
<td>n/a</td>
<td>$M_{\text{max}}$ ratio between pile groups ($L_m/L_s = 200/500$ and $400/300$)</td>
<td>0.25</td>
<td>n/a</td>
<td>n/a</td>
</tr>
</tbody>
</table>
5.7 The effect of pile diameter

The effect of the pile diameter was investigated by comparing two 2 × 2 pile group tests \( (L_m/L_s = 200/500) \) with different pile diameters of 32 mm and 50 mm, respectively. Only the 2 × 2 pile group tests were conducted with two different pile diameters, hence, the effect of the pile diameter was only investigated on this pile group. Figure 5.20(a) shows that the \( M_{\text{max}} \) on pile A having a 50 mm diameter, which is higher than that having a 32 mm diameter. As discussed in the previous subsection (5.6.3), the negative bending moment on the piles (A and B) (Figure 5.20(b)) is due to the restraint provided by the pile cap and the interaction between the piles in the pile groups. Table 5.6 shows the different ratios of the \( M_{\text{max}} \) against the \( M_{\text{max}} \). On pile A, with a 50 mm diameter, the \( M_{\text{max}} \) is almost equal to the \( M_{\text{max}} \). Also, on piles A and B, the \( M_{\text{max}} \) and \( M_{\text{max}} \) increase with the pile diameter (or pile stiffness). Thus, the piles with a larger diameter are subjected to a higher pressure from the moving soil; as a result of this pressure, a higher load is acting on the piles which, in turn, induces a larger moment on the piles (this behaviour only holds provided that the pile is relatively stable).
Figure 5.20 The effect of pile diameter on the bending moment profile of 2 × 2 pile group

Table 5.6 Summary of the effect of pile diameter on the piles (\(w_s = 60\) mm)

<table>
<thead>
<tr>
<th>Pile A (coloured in black)</th>
<th>(M_{\text{max}}) (Nmm)</th>
<th>(M_{\text{max}}) (Nmm)</th>
<th>Ratio (M_{\text{max}}/M_{\text{max}})</th>
<th>Pile B (coloured in black)</th>
<th>(M_{\text{max}}) (Nmm)</th>
<th>(M_{\text{max}}) (Nmm)</th>
<th>Ratio (M_{\text{max}}/M_{\text{max}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>32 mm</td>
<td>9,943</td>
<td>-5,539</td>
<td>1.80</td>
<td>32 mm</td>
<td>8,711</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>50 mm</td>
<td>14,952</td>
<td>-15,888</td>
<td>0.94</td>
<td>50 mm</td>
<td>14,976</td>
<td>-8,016</td>
<td>1.87</td>
</tr>
<tr>
<td>(M_{\text{max}}) ratio between pile groups ((d = 32) mm and (50) mm)</td>
<td>0.66</td>
<td>0.35</td>
<td>n/a</td>
<td>(M_{\text{max}}) ratio between pile groups ((d = 32) mm and (50) mm)</td>
<td>0.58</td>
<td>n/a</td>
<td>n/a</td>
</tr>
</tbody>
</table>
5.8 The effect of pile spacing

The effect of pile spacing was investigated for both the piles in a row (1 × 2) and in a square (2 × 2) arrangement. Two pile spacings of 3d and 5d, were varied for the tests, for each pile group. These tests were compared according to the following pile group:

- 1 × 2 pile group at $L_m/L_s = 200/500$ (d = 32 mm), Subsection 5.8.1
- 2 × 2 pile group at $L_m/L_s = 200/500$ (d = 32 mm), Subsection 5.8.2
- 2 × 2 pile group at $L_m/L_s = 200/500$ (d = 50 mm), Subsection 5.8.3
- 2 × 2 pile group at $L_m/L_s = 400/300$ (d = 32 mm), Subsection 5.8.4.

5.8.1 Effect of spacing on piles in a row (1 × 2, $L_m/L_s = 200/500$, d = 32 mm)

For comparative purposes, the single pile test, presented in Chapter 4, is again presented here to identify the effect of the pile spacing on the 1 × 2 pile group, with 32 mm diameter piles at $L_m/L_s = 200/500$. As previously noted in Section 5.3, the response of the piles in this pile group is similar, for this reason, only pile A is discussed. Figures 5.21(a), (b) and (c) show the bending moment profiles, the soil reaction profile and the $M_{max}$ against soil movement plot, respectively. A comparison between the pile responses shows the following key features:

1) the $M_{max}$ for the pile group with 3d and 5d spacings are 6.5 % and 1.8 %, respectively, higher than the single pile

2) the maximum positive soil reactions (in the $L_m$), for the pile group with 3d and 5d spacings are 41.9 % and 29.5 %, respectively, higher than that obtained from the single pile test

3) the maximum pile deflection at the soil surface for the single pile and the pile group of 3d and 5d spacings are 6.7 mm, 12.3 mm and 4.71 mm, respectively (Figure 5.2(d)). These values are less than 25% of the total applied soil movement on the pile ($w_s = 60$mm)

4) the deflection profile of the single pile shows, mainly, the rotation about the 650 mm depth. In contrast, for the pile groups, the deflection profiles of pile A show, mainly, a translation with a slight rotation for the 3d spacing, and both a translation and a rotation for the 5d spacing.
Figure 5.21 The effect of pile spacing on the response of $1 \times 2$ pile group
5.8.2 Effect of spacing on piles in a square (2 × 2, \( L_m/L_s = 200/500 \), \( d = 32 \text{ mm} \))

The effect of the pile spacing was investigated for the 2 × 2 pile group with 32 mm diameter piles at \( L_m/L_s = 200/500 \). Figure 5.22 shows the pile response in term of the bending moment and the shear force profiles from the two tests conducted with 3d and 5d pile spacings, respectively. The following pile behaviours are identified:

1) at \( w_s = 60 \text{ mm} \), pile A with the 5d spacing recorded a higher negative and positive bending moments, as compared to pile A with the 3d spacing

2) the shapes of the bending moment profiles also indicate that pile A, for both tests, have a common deformation pattern

3) pile B, from the test with the 3d spacing, has different a deformation pattern when compared to pile B with the 5d spacing

4) the negative bending moments (above \( L_m \)) are recorded for the tests on piles A and B with 5d spacing. As shown in Figure 5.22(b), only pile A, with a 3d spacing, shows negative bending moments above \( L_m \)

5) the difference in the magnitude of the negative bending moments indicates that the increase in the pile spacing (from 3d to 5d) corresponds to a larger restraint provided by the pile cap onto the piles. Not surprisingly, with the larger restraint, the deflection measured at the pile cap level reduces from 9.41 mm to 8.27 mm, as the pile spacing increases from 3d to 5d (see Appendix C for the pile deflection profiles).
The responses at the pile cap (500 mm above the soil surface) of the pile group are shown in Figure 5.23(a). It was expected that, for a perfect symmetrical loading (applied soil movement) and for the pile arrangement in the shear box, that the results would be similar between both front piles (piles A), while the responses on the back piles (piles B) should also be similar. However, it is difficult to achieve a perfectly symmetrical condition, especially because of the uncontrollable way the soil moves around the pile.
Chapter 5: Pile group tests

Figure 5.23 Forces and moments in piles A and B when subjected to lateral soil movement

The role of vertical forces and pile spacing to increase restraint on the pile group

The forces inflicted on a pile group subjected to soil movement can be seen in Figure 5.23(b). Generally, there are three load components, acting on each pile, and connected to the pile cap. For pile A, these loads are a vertical force ($V_{A,cap}$), a lateral force or a shear force ($H_{A,cap}$), and a moment ($M_{A,cap}$). Similarly, there are three load components acting on pile B ($V_{B,cap}$, $H_{B,cap}$ and $M_{B,cap}$). The degree of restraint by pile A can be said to depend on the resistance given by the adjacent pile B, where the $M_{A,cap} = f(V_{B,cap}, \text{pile spacing})$ and the $M_{B,cap} = f(V_{A,cap}, \text{pile spacing})$. Hence, the restraining moment on piles A and B depends on the function of: 1) the vertical forces $V_{A,cap}$ and $V_{B,cap}$; and 2) the pile spacing. As can be seen from Table 5.6, the $M_{\max}$ and $M_{\max}$ increase with the pile spacing.
Table 5.7 Summary of the effect of the pile spacing on the piles (\(w_s = 60\) mm)

<table>
<thead>
<tr>
<th>Pile A (coloured in black)</th>
<th>M_{\text{max}} (Nmm)</th>
<th>M_{\text{max}} (Nmm)</th>
<th>Ratio of (M_{\text{max}}/M_{\text{max}})</th>
<th>Pile B (coloured in black)</th>
<th>M_{\text{max}} (Nmm)</th>
<th>M_{\text{max}} (Nmm)</th>
<th>Ratio of (M_{\text{max}}/M_{\text{max}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>3d spacing</td>
<td>9,943</td>
<td>-5,251</td>
<td>1.89</td>
<td>3d spacing</td>
<td>8,712</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>5d spacing</td>
<td>12,380</td>
<td>-10,062</td>
<td>1.23</td>
<td>5d spacing</td>
<td>12,013</td>
<td>-5,163</td>
<td>2.33</td>
</tr>
<tr>
<td>M_{\text{max}} ratio between pile groups (3d and 5d spacings)</td>
<td>0.80</td>
<td>0.52</td>
<td>n/a</td>
<td>M_{\text{max}} ratio between pile groups (3d and 5d spacings)</td>
<td>0.73</td>
<td>n/a</td>
<td>n/a</td>
</tr>
</tbody>
</table>

The role of the reaction forces from the adjacent pile to increase the restraint on the pile group, as follows:

1) above the soil surface the shear forces \(H_{A,\text{surface}}\) and \(H_{B,\text{surface}}\) remain constant along the piles until the opposite direction \(H_{A,\text{cap}}\) and \(H_{B,\text{cap}}\) at the pile cap level acts as balancing forces, respectively. The relation between these forces can be summarised as follow:

\[|H_{A,\text{surface}}| = |H_{A,\text{cap}}| \quad \text{and} \quad |H_{B,\text{surface}}| = |H_{B,\text{cap}}|\]

2) pile A is loaded laterally at the pile cap level with \(H_{A,\text{cap}}\), acting in the opposite direction of the soil movement

3) pile B is loaded laterally at the pile cap level with \(H_{B,\text{cap}}\), acting in the same direction as the soil movement.

The shear force profiles from Figures 5.22(c) and (d) show the shear forces at the soil surface for piles A and B of 3d spacing are \(H_{A,\text{surface}} = 12.47\) N and \(H_{B,\text{surface}} = -7.11\) N, respectively. The shear forces at the surface for piles A and B, of 5d spacing, are \(H_{A,\text{surface}} = 18.62\) N and \(H_{B,\text{surface}} = -22.94\) N, respectively. These results show that:

1) the shear forces increase with the pile spacing

2) both piles A and B are laterally loaded at the pile cap level, with the lateral forces acting in the opposite directions.
5.8.3 Effect of spacing on piles in a square (2 × 2, \(L_m/L_s = 200/500\), \(d = 50\) mm)

The results of two tests on the 50 mm diameter pile, with spacings of 3d and 5d, respectively, can be seen in Figures 5.24(a) and (b), with the bending moment profiles at \(w_s = 60\) mm for piles A and B, respectively. The bending moment profiles for pile A, are very similar in shape, and the ratio of the \(M_{\text{max}}\), between the pile group with the 3d and 5d pile spacings, are equal to 1.08 (Table 5.8). On the other hand, for pile B, the \(M_{\text{max}}\) ratio between the pile group with the 3d and 5d spacings, is 0.54. A higher \(M_{\text{max}}\) in both piles, with the 5d spacing, highlights the increase of the pile restraint with the increase of the pile spacing.

The soil reactions on pile A, from both tests (3d and 5d), are higher than the soil reactions from pile B (Figures 5.24(c) and (d)). This difference shows that pile A, which is located directly “facing” the applied soil movement has “shadowing” effect on pile B, where the load developed from the soil passing the pile group is first taken by pile A before a smaller portion of the load is transferred to pile B. The differences in soil reactions between the tests (3d and 5d spacings) are 17.0 % and 26.3 % for piles A and B, respectively.

Load transfer between piles A and B as \(w_s\) increases

For the purpose of this discussion, the focus is on the pile group with the 5d pile spacing. The soil reaction profiles, at the different level of \(w_s\), and the indication of the load transfer between piles are plotted in Figure 5.25. Table 5.9 describes the mechanism of the load transfer. Importantly, the “shadowing” effect on Pile B is only noted on the 2 × 2 pile group tests at \(L_m/L_s = 200/500\).
Figure 5.24 The effect of pile spacing on the bending moment profile of 2×2 pile group

Table 5.8 Summary of the effect pile spacing on the piles (ws = 60 mm)

<table>
<thead>
<tr>
<th>Pile A (coloured in black)</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>Ratio of $M_{\text{max}}/M_{\text{max}}$</th>
<th>Pile B (coloured in black)</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>Ratio of $M_{\text{max}}/M_{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3d spacing</td>
<td>14,952</td>
<td>-19,392</td>
<td>0.77</td>
<td>3d spacing</td>
<td>14,976</td>
<td>-8,016</td>
<td>1.87</td>
</tr>
<tr>
<td>5d spacing</td>
<td>13,560</td>
<td>-23,208</td>
<td>0.58</td>
<td>5d spacing</td>
<td>27,648</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>$M_{\text{max}}$ ratio between pile groups (3d and 5d spacings)</td>
<td>1.08</td>
<td>0.84</td>
<td>n/a</td>
<td>$M_{\text{max}}$ ratio between pile groups (3d and 5d spacings)</td>
<td>0.54</td>
<td>n/a</td>
<td>n/a</td>
</tr>
</tbody>
</table>
Figure 5.25 Soil reaction at different $w_s$ indicating load transfer between piles A & B
### Table 5.9 Summary of load transfer between piles A and B

<table>
<thead>
<tr>
<th>$w_s$ (mm)</th>
<th>Pile A</th>
<th>Pile B</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>• Soil pressure (passive pressure) build up due to the moving soil on the Pile A: Point a&lt;br&gt;• Also creating “shadowing” effect on the soil movement to Pile B: Point c</td>
<td>• Laterally loaded (active) by Pile A as it deflects: Point b&lt;br&gt;• Soil pressure (resisting the lateral load) in L_m are in opposite direction with to that on Pile A. Point e</td>
</tr>
<tr>
<td>30</td>
<td>• Passive pressure continues to increase as $w_s$ increases: Point a&lt;br&gt;• Creating “shadowing” effect on the soil movement to Pile B continues: Point d</td>
<td>• Laterally loaded (active) by Pile A as it deflects and the load increases as $w_s$ increases: Point c&lt;br&gt;• Passive pressure on Pile B begins to build up as a result of soil movement (“shadowing” effect reached its peak): Point e&lt;br&gt;• Lower net pressure (active and passive) recorded in L_m: Points d, e</td>
</tr>
<tr>
<td>60</td>
<td>• Passive pressure on Pile A reaches its limiting value: Point a&lt;br&gt;• “Shadowing” effect reached its limiting value: Point c&lt;br&gt;• Soil movement begins to squeezes between and flows Piles A: Point e</td>
<td>• Lateral load from Pile A reached its limiting value: Point c&lt;br&gt;• Passive pressure continues to build up on Pile B (at L_m): Point c</td>
</tr>
<tr>
<td>120</td>
<td>• Passive pressure reaches a limiting value: Point a</td>
<td>• Both active and passive pressures reach a limiting value: Points a, c</td>
</tr>
</tbody>
</table>
5.8.4 Effect of spacing on piles in a square (2 × 2, \(L_m/L_s = 400/300\), \(d = 32\) mm)

The effect of pile spacing on the pile responses for the test with \(L_m/L_s = 400/300\) is more significant when compared to the tests with \(L_m/L_s = 200/500\), previously reported in Subsections 5.8.2 and 5.8.3. Figure 5.26 shows the bending moment profiles for the two tests (\(L_m/L_s = 400/300\)), with 3d and 5d spacings. There are errors in pile B of the test with 5d spacing (Subsection 5.5.2.2), nevertheless, for the test with 3d spacing, the bending moment profiles between piles A and B are similar in shape. The difference in the \(M_{+\max}\) between piles A and B is only 0.16 %.

The differences and the ratios of the \(M_{-\max}\) and the \(M_{+\max}\) are tabulated in Table 5.10. The results indicate that, by increasing the spacing from 3d to 5d, the response of pile A:

1) increases in \(M_{-\max}\) from -6,178 Nmm to -9,950 Nmm (or 61 %)
2) reduces in \(M_{+\max}\) from 31,922 Nmm to 6,537 Nmm (or 388 %)
3) generally, means that the shape of the bending moment profiles are both negative and positive values in \(L_m\) and \(L_s\), respectively.

A lower \(M_{+\max}\) at a higher pile spacing could indicate the increased ability of the piles, as a group, to resist the imposed soil movement in a “push-pull” manner, rather than relying solely on its bending resistance. On the other hand, a higher \(M_{-\max}\) at a 5d pile spacing indicates that the restraint provided by the pile cap increases with the pile spacing.
Figure 5.26 The effect of pile spacing on the bending moment profile of 2 × 2 pile group

Table 5.10 Summary of the effect of the pile spacing on the piles (ws = 60 mm)

<table>
<thead>
<tr>
<th>Pile A ( coloured in black )</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>Ratio of $M_{\text{max}}/M_{\text{max}}$</th>
<th>Pile B ( coloured in black )</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>$M_{\text{max}}$ (Nmm)</th>
<th>Ratio of $M_{\text{max}}/M_{\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3d spacing</td>
<td>31,922</td>
<td>-6,178</td>
<td>5.17</td>
<td>3d spacing</td>
<td>35,054</td>
<td>-2,717</td>
<td>12.90</td>
</tr>
<tr>
<td>5d spacing</td>
<td>6,537</td>
<td>-9,950</td>
<td>0.66</td>
<td>5d spacing</td>
<td>12,899</td>
<td>n/a</td>
<td>n/a</td>
</tr>
<tr>
<td>$M_{\text{max}}$ ratio between pile groups (3d and 5d spacings)</td>
<td>4.88</td>
<td>0.62</td>
<td>n/a</td>
<td>$M_{\text{max}}$ ratio between pile groups (3d and 5d spacings)</td>
<td>2.72</td>
<td>n/a</td>
<td>n/a</td>
</tr>
</tbody>
</table>
5.8.5 Summary on the effect of pile spacing

In the preceding subsections, the effect of the pile spacing on the different pile groups has been discussed in detail. Figure 5.27 presents the $M_{\text{max}}$ and $r_{\text{max}}$ obtained from these pile groups.

![Graph](image_url)

(a) $M_{\text{max}}$ against pile spacing

(b) $r_{\text{max}}$ against pile spacing

Figure 5.27 The effect of pile spacing on the pile response in term of $M_{\text{max}}$ and $r_{\text{max}}$
From these figures, and with reference to the preceding subsections, the following summaries can be drawn:

(a) 1 × 2 pile group with \( L_m/L_s = 200/500 \) (Subsection 5.8.1)

As the pile spacing increases, the difference in \( M_{\max} \) is less significant when compared to the difference in \( r_{\max} \). Both the \( M_{\max} \) and the \( r_{\max} \) reduce as the pile spacing increases from 3d to 5d. At the 5d pile spacing, the difference between the piles in the pile group and the single pile is less significant.

(b) 2 × 2 pile group with \( L_m/L_s = 200/500 \) (Subsections 5.8.2 and 5.8.3)

On piles A, the \( M_{\max} \) and \( M_{\min} \) increase with the pile spacing, while on pile B, only the \( M_{\max} \) increases with the pile spacing. However, for the pile group with the larger pile diameter, these increments tend to reduce, especially for pile A.

For any of the given pile spacings, the difference in the \( M_{\max} \) between piles A and B is insignificant. In contrast, the difference in the \( r_{\max} \) between piles A and B is significant, where \( r_{\max} \) on pile A is up to 1.6 times that on pile B.

(c) 2 × 2 pile group with \( L_m/L_s = 400/300 \) (Subsection 5.8.4)

The effect of the pile spacing on the 2 × 2 pile group with \( L_m/L_s = 400/300 \) is differed from the same pile group with \( L_m/L_s = 200/500 \), namely:

- on pile A, the \( M_{\max} \) and \( r_{\max} \) reduce as the pile spacing increases (see Subsection 5.8.4 for possible causes of these reductions)
- on any given pile spacing, the difference in the \( r_{\max} \) between piles A and B is less significant (below 8%).
5.9 The effect of pile group arrangement

By understanding the effect of the pile group arrangement, it is useful to compare the piles in a pile group with that of the single pile. The method used here was previously introduced by Poulos and Davis (1980) for the analysis of the laterally loaded piles, and later adopted by Chen (1994) for piles subjected to lateral soil movement. The maximum bending moment of the piles (in a group) was compared to that of the “standard” single pile test (“standard” being having the same $L_m/L_s$, sand and pile properties), at the same amount of the lateral soil movement ($w_s$). This comparison can be expressed in the following ratio:

$$F_m = \frac{M_{\text{max},i}}{M_{\text{max},s}}$$

where $F_m$ : group factor in term of maximum bending moment  
$M_{\text{max},i}$ : maximum bending moment of Pile ‘i’ in a pile group  
(i refers to either pile A or B in this thesis)  
$M_{\text{max},s}$ : maximum bending moment of the “standard” single pile.

A further comparison was made between the maximum soil reaction ($r_{\text{max}}$) recorded on the piles in a pile group and that of the “standard” single pile test. In this case, the group factor, in terms of $r_{\text{max}}$, can be expressed as follows:

$$F_r = \frac{r_{\text{max},i}}{r_{\text{max},s}}$$

where $F_r$ : group factor in terms of the maximum soil reaction  
$r_{\text{max},i}$ : the maximum soil reaction of pile ‘i’ in a pile group  
(i refers to either pile A or B in this thesis)  
$r_{\text{max},s}$ : the maximum soil reaction of the “standard” single pile test.

The effect of the pile group arrangements was investigated in the tests with the moving and the stable layers of 200 mm and 500 mm, respectively. Figure 5.28 shows the bending moment profiles of the single pile test, and those of the pile group tests with: 1) piles in a line; 2) piles in a row; and 3) piles in a square. As previously
discussed, the bending moment on pile B of the $2 \times 2$ pile group was not recorded correctly, hence, it was not included in Figure 5.28(b) for comparison.

The comparative results from the pile group tests are presented below:

1) the highest $M_{\text{max}}$ is on pile A of the $1 \times 2$ pile group
2) the lowest $M_{\text{max}}$ is on both pile A of the $2 \times 1$ pile group
3) the highest $M_{\text{max}}$ is found on pile A of the $2 \times 1$ pile group
4) on pile A, the $M_{\text{max}}$ on the $2 \times 2$ pile group is approximately half of that recorded on the $2 \times 1$ pile group
5) both the single pile and $1 \times 2$ pile group (piles A and B) tests recorded the $M_{\text{max}}$ close to zero at the soil surface
6) both the single pile and $1 \times 2$ pile group (piles A and B) tests have bending moment profiles that are similar in terms of shape and magnitude
7) the shape of the soil reaction profiles are similar for all the tests
8) on each pile, the highest $r_{\text{max}}$ is from the $1 \times 2$ pile group
9) on each pile, the lowest $r_{\text{max}}$ is from the $2 \times 1$ pile group.
Table 5.11 shows the $F^m$ and $F^r$ ratios for piles in each pile group, namely that:

1) the $1 \times 2$ pile group (pile B) are the most similar to the single pile, with $F^m$ and $F^r$ ratios of 0.95 and 1.12 (in $L_m$), respectively

2) the $2 \times 1$ pile group shows the highest differences from the single pile, with $F^m$ and $F^r$ ratios of 0.25 and 0.45 (in $L_s$), respectively.

Figure 5.28 The effect of pile group arrangement on the pile response
Table 5.11 Comparison between different pile group arrangements with single pile
\((w_s = 60 \text{ mm})\)

<table>
<thead>
<tr>
<th>Pile A (in black)</th>
<th>(r_{\text{max}/r_{\text{max}s}})</th>
<th>(M_{\text{max}/M_{\text{max}s}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1 \times 2)</td>
<td>0.48/0.34 (= 1.41)</td>
<td>26,846/25,211 (= 1.06)</td>
</tr>
<tr>
<td>(2 \times 1)</td>
<td>0.23/0.34 (= 0.68)</td>
<td>6,282/25,211 (= 0.25)</td>
</tr>
<tr>
<td>(2 \times 2)</td>
<td>0.25/0.34 (= 0.74)</td>
<td>9,942/25,211 (= 0.39)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Pile B (in black)</th>
<th>(r_{\text{max}/r_{\text{max}s}})</th>
<th>(M_{\text{max}/M_{\text{max}s}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1 \times 2)</td>
<td>0.38/0.34 (= 1.12)</td>
<td>23,841/25,211 (= 0.95)</td>
</tr>
<tr>
<td>(2 \times 1)</td>
<td>0.22/0.34 (= 0.65)</td>
<td>12,484/25,211 (= 0.50)</td>
</tr>
<tr>
<td>(2 \times 2)</td>
<td>n/a</td>
<td>n/a</td>
</tr>
</tbody>
</table>

\(r_{\text{max}}\), \(r_{\text{max}s}\), \(M_{\text{max}}\), \(M_{\text{max}s}\)
5.10 Limiting soil pressure profile

The soil reaction profiles for all the pile group tests (with and without an axial load) were presented in the earlier sections (the individual pile response is detailed in Appendix C). These profiles are summarised to identify the maximum soil reaction \( r_{\text{max}} \) on the piles in a group. The soil reaction on the piles was investigated separately in the moving layer \( (L_m) \) and in the stable layer \( (L_s) \). In \( L_s \), the stable soil provided resistance to the whole pile. Thus, in \( L_s \), the pile is similar to that of the laterally loaded piles.

The maximum soil reaction, \( r_{\text{max}} \), in both the \( L_m \) and \( L_s \) layers were the maximum values obtained from the soil reaction profile of the individual piles in a pile group test, as shown in Figure 5.29.

![Typical soil reaction profiles showing the \( r_{\text{max}} \) in \( L_m \) and \( L_s \)](image)

As both 32 mm and 50 mm diameter piles were used, it appears to be more appropriate to present the maximum soil reaction in the form of the maximum soil pressure, \( p_{\text{max}} \) (where \( p_{\text{max}} = \frac{r_{\text{max}}}{d} \)), for the purpose of comparison.

The maximum soil pressure \( (p_{\text{max}}) \) acting on the piles (A and B) in the pile group tests at \( w_s = 60 \text{ mm} \) are plotted in Figure 5.30. Also shown are three limiting soil pressure profiles: 1) \( K_p\gamma z \), (Rankine's passive earth pressure); 2) \( 3K_p\gamma z \), (Broms 1964); and 3) \( K_p^2\gamma z \), (Barton 1982). \( K_p \) is the coefficient of passive earth pressure; \( \gamma \) is the sand unit weight (constant in this study); and \( z \) is the depth below soil surface. In general, the
maximum soil pressure recorded from the pile group tests did not exceed the limiting soil pressure described by Barton (1982).

5.10.1 Maximum soil pressure in the moving layer (Lm)

Figure 5.30 shows the maximum pressure recorded from the tests with the L_m/L_s of 200/500. The locations of the \( p_{\text{max}} \) are at a depth of 60 mm for all the tests (depending on the pile diameter, this depth is equivalent to 1.2d to 1.875d). Piles A and B of the 2 \( \times \) 2 pile group are plotted in Figure 5.30(a) to highlight the difference in the maximum soil pressure, \( p_{\text{max}} \), on these piles. Pile A recorded a higher soil pressure when compared to pile B. The magnitude of the soil pressure on pile A generally falls between \( 3K_p \gamma z \) and \( K_p^2 \gamma z \). On the other hand, the \( p_{\text{max}} \) on pile B are generally below \( K_p \gamma z \). This outcome suggests that pile A, being located in front of the moving soil, is subjected to higher soil pressure, while creating a “shadowing” effect on pile B. This “shadowing” effect reduces the magnitude of the soil movement on pile B, which in turn reduces the \( p_{\text{max}} \) on pile B (see Subsection 5.8.3 for a detailed discussion on the soil-pile interaction).

Figure 5.30(b) shows the results of the 1 \( \times \) 2 and 2 \( \times \) 1 pile group tests. Pile A and B are not plotted separately in the figure since it has been shown, previously, that there is no significant difference in \( r_{\text{max}} \) (or \( p_{\text{max}} \)) between the piles in these pile groups. The \( p_{\text{max}} \) are generally less than \( K_p^2 \gamma z \); while, for 87.5% of these tests the \( p_{\text{max}} \) are less than \( 3K_p \gamma z \).
Figure 5.30 Maximum soil pressure in $L_m$ for tests with $L_m/L_s = 200/500$

Figure 5.31 Shows the $p_{max}$ recorded for the tests with $L_m/L_s = 400/300$. The tests on the $2 \times 2$ pile group are plotted in Figure 5.31(a). When compared to the tests with $L_m/L_s = 200/500$ (Figure 5.30(a)), piles A and B show no clear difference in the $p_{max}$ when the $L_m/L_s = 400/300$. The $p_{max}$ on these piles are lower than $K_p \gamma z$, except for one test where pile B shows the $p_{max}$ higher than $K_p \gamma z$ by 6.3%. The locations of the $p_{max}$ on pile A are generally at a depth of 260 mm, with one exception at the depth of 160 mm. On the other hand, the $p_{max}$ on pile B are consistently located at a depth of 160 mm.

Figure 5.31(b) shows the $p_{max}$ on the $1 \times 2$ pile group located at a depth of 260 mm; the values fall between $K_p \gamma z$ and $3K_p \gamma z$. The $p_{max}$ on the $2 \times 1$ pile groups are above $3K_p \gamma z$, with 50% of tests recording values higher than $K_p \gamma z$, by an average of 4.4%. The locations of the $p_{max}$ range from 60 mm to 260 mm.
5.10.2 Maximum soil pressure in the stable layer ($L_s$)

As shown in Figure 5.29, the $r_{max}$ in the stable layer ($L_s$) is taken from the maximum value of the soil reaction profile (in $L_s$) of the piles (A and B) in a pile group.

Figure 5.32(a) shows the $p_{max}$ obtained from the 2 × 2 pile group with $L_m/L_s = 200/500$. The $p_{max}$ in $L_s$ are below $K_p \gamma z$, possibly indicating, for the tests with $L_m/L_s = 200/500$ (where $L_m < L_s$), that the load from the moving soil (in $L_m$) only requires a portion of the limiting pressure (in $L_s$) to provide resistance so that the overall stability of the piles and the pile group could be achieved.

The $p_{max}$ in pile A is generally higher than that of pile B. This difference is also noted from the $p_{max}$ obtained in the $L_m$ (Figure 5.30(a)). The results from the tests have shown that, as the magnitude of the soil pressure generated in $L_m$ by the moving soil increases, so does the magnitude of the resistance pressure generated in $L_s$. As previously shown in Subsection 5.10.1, where the $p_{max}$ in pile A is higher than that of pile B in $L_m$, not surprisingly, the $p_{max}$ in pile A is also higher than that of pile B in $L_s$. All except two tests (see Figure 5.32(a)), show that $p_{max}$ are located at a depth of 460 mm. This depth is approximately 5.2d to 8.1d below the sliding depth.

Figure 5.32(b) shows the $p_{max}$ obtained from the 1 × 2 and 2 × 1 pile groups. Generally, $p_{max}$ on all the tests are below $K_p \gamma z$, with the exception on one test (1 × 2), which exceeds $K_p \gamma z$ by 11%. As mentioned previously, the low $p_{max}$ ($\leq K_p \gamma z$) shows that the limiting pressure in $L_s$ is not fully mobilised by the load from the moving soil.
Chapter 5: Pile group tests

Figure 5.32 Maximum soil pressure in $L_s$ for tests with $L_m/L_s = 200/500$

Figure 5.33 shows the $p_{\text{max}}$ in $L_s$ for the $2 \times 2$ pile group with $L_m/L_s = 400/300$. For most tests, the $p_{\text{max}}$ fall between $K_p \gamma z$ and $3K_p \gamma z$. These $p_{\text{max}}$ are higher when compared to that obtained from the $2 \times 2$ pile group with $L_m/L_s = 200/500$ (Figure 5.28(a)). Furthermore, the $p_{\text{max}}$ on piles A and B with $L_m/L_s = 400/300$ did not show a distinctive difference, as noted in the tests with $L_m/L_s = 200/500$. The depth of $p_{\text{max}}$ is consistently at 460 mm for piles A and B.

Figure 5.33(b) shows the $p_{\text{max}}$ in $L_s$ for $1 \times 2$ and $2 \times 1$ pile groups with $L_m/L_s = 400/300$. The $p_{\text{max}}$ for the pile group of $1 \times 2$ fall between $3K_p \gamma z$ and $K_p^2 \gamma z$, while, the $p_{\text{max}}$ for the $2 \times 1$ pile group have a higher magnitude, and generally ranges in between $K_p \gamma z$ and $K_p^2 \gamma z$, with one test exceeding $K_p^2 \gamma z$ by 8.6 %. The depth of the $p_{\text{max}}$ is found to be at 460 mm for all the tests.
Figure 5.33 Maximum soil pressure in $L_s$ for tests with $L_{mf}/L_s = 400/300$
5.11 On $M_{\text{max}}$ against $S_{\text{max}}$ relationship on the pile group tests

For a pile group test, with a set of input parameters, such as $L_{\text{m}}/L_{\text{s}}$, pile diameter, axial load and pile spacing, it may be expected that a single line would give a good fit to the $M_{\text{max}}$ against the $S_{\text{max}}$ relationship, with a corresponding high value of the coefficient of determination $r_d^2$. Figures 5.34(a), (b) and (c) show the $M_{\text{max}}$ against the $S_{\text{max}}$ for all the pile group tests compiled according to the $1 \times 2$, the $2 \times 1$ and $2 \times 2$ pile groups, respectively.

When all the pile tests are plotted on their respective group, it can be seen that the relationship $M_{\text{max}}$ against $S_{\text{max}}$ holds a relatively high value of the coefficient of determination $r_d^2$ of 0.66 ~ 0.93. Additionally, the $r_d^2$ of 0.66, obtained from the $2 \times 2$ pile group, is the lowest when compared to the other pile groups. This lower $r_d^2$ indicates that the $L_{\text{m}}/L_{\text{s}}$, pile diameter, axial load and pile spacing have a greater influence on the pile group response when compared to the other pile groups ($1 \times 2$ and $2 \times 1$).

A detailed analysis of the $M_{\text{max}}$ against $S_{\text{max}}$ relationship was undertaken by Guo and Qin (2009) and Guo (2009) for the single pile and pile groups tests, respectively. In their investigation, it was concluded that $M_{\text{max}}$ was largely linearly related to the sliding force (or $S_{\text{max}}$) in a relationship $M_{\text{max}} = S_{\text{max}}L/2.8$. With this relationship a simple solution was also proposed, which offered a good prediction on a number of case studies. The simplified solution was useful for the preliminary estimation of the $M_{\text{max}}$ for pile group design.
Chapter 5: Pile group tests

Figure 5.34 $M_{\text{max}}$ against $S_{\text{max}}$ relationship on the pile group tests
5.12 Other findings

A number of other findings were also identified as part of the experimental program, namely:

(1) lateral force to cause soil movement, Subsection 5.12.1
(2) pile penetration resistance, Subsection 5.12.2.

5.12.1 Lateral force to cause soil movement

The forces in the lateral and the vertical hydraulic jacks while pushing the aluminium frames (L_m) and driving the piles into the shear box, respectively, were only recorded at the later stage of the pile group experimental program. Figure 5.35 shows the lateral force per pile (averaged over the piles in a group) against the laminar frames movement (see Chapter 3, Subsection 3.2.1), which was recorded as w_s. The piles in the 2 × 2 pile group recorded a maximum lateral force when the movement is >60 mm, while, on the other pile groups (1 × 2 and 2 × 1), the lateral forces reaches their maximum at smaller frame movement of >10 mm. The forces here are only for the 32 mm diameter pile. For the piles at L_m/L_s = 200/500, with all the pile spacings (3d, 5d, and 10d), the maximum lateral force per pile is approximately 2 kN. At L_m/L_s = 400/300, the pile group of 2 × 1 and 2 × 2 recorded lateral forces per pile of 4.3 kN and 6.0 kN, respectively. No direct comparison can be made between the tests with L_m/L_s of 200/500 and 400/300, as the data are from different pile groups (1 × 2 and 2 × 2). However, the current results suggest that, as L_m increases from 200 mm to 400 mm, the lateral force per pile increases by approximately 2 to 3 times. However, there is no significant difference in the recorded lateral force per pile between the tests conducted with and without the axial load applied onto the pile cap.
Figure 5.35 Total lateral force vs. frame movement
5.12.2 Pile penetration resistance

The driving force was recorded by using the mechanical pressure gauge attached to the hydraulic pump. Figure 5.36 shows the driving force, per pile of the pile group, into the shear box. The force per pile was the average force, over the piles in a group, required to push the pile into the required depth of 700 mm inside the shear box. The force (per pile) to drive the $2 \times 2$ pile group is 21.4% higher than the $1 \times 2$ pile group with the spacing of 3d. For the $1 \times 2$ pile group, the driving force per pile reduces by 12 % as the pile spacing increases from 3d to 10d. Similarly, for the $2 \times 2$ pile group, the driving force per pile decreases by 21.4 % as the pile spacing increases from 3d to 5d. At the 5d spacing, both the $1 \times 2$ and $2 \times 2$ pile groups recorded the same value for the driving force per pile at a depth of 700 mm.

![Figure 5.36 Driving resistance during installation](image-url)
5.13 Limitation on the pile group tests

Size of the shear box

With the current size of the shear box being 1.0 m × 1.0 m (plan dimensions), the boundary effect between the pile and the shear box wall was the limitation on the maximum number of piles to be tested in a group; this limit was the 2 × 2 pile group with a maximum pile spacing of 5d.

Instrumentation

A total of ten strain gauges were attached along each pile for a pile group test, with a total of two active instrumented piles per test. With the current available strain gauge channels (24 in total), the gauges on the pile were spaced at 100 mm, centre to centre. One possible improvement could be to increase the capacity of the data acquisition system with more strain gauge channels, as additional strain gauges could be installed on the pile section, above the soil surface and on the pile cap (see Figure 3.24 for strain gauge locations). These improvements would enable an additional bending moment on the pile section, above the soil surface and the pile cap, to be measured. As previously shown in the 1 × 2 and 2 × 2 pile group tests, those piles in symmetrical position in a group can still have differences in their responses. With additional stain gauges, all the piles in a group can be instrumented and the differences between the piles in symmetrical positions can be accurately identified.

The use of a full-bridge connection with four strain gauges at each section of the pile could be another improvement. This connection acts effectively as a load cell which enables axial stress distribution to be investigated and measured. It is envisaged that when axial load is applied on the pile head, the axial stress is highest at the pile head and reduces towards the pile toe.
Pile installation effect

One major effect of the pile installation is its influence on the change of density of sand around the pile group after the piles in a group are driven into the shear box. Unfortunately, the extent of this change is difficult to measure.

In the current apparatus setup, all possible care was taken to ensure the verticality of the pile while pushing the pile into place in the shear box. However, the piles were occasionally found to be slightly off the vertical position. When the axial load was added, a slight deviation of the pile from the perfectly vertical had a significant effect on the pile deflection and bending moment.

5.14 Conclusions

In the current study, a number of model tests were carried out on the pile group in arrangements of 1 × 2 (in a row), 2 × 1 (in a line) and 2 × 2 (in a square). Additionally, only the rectangular soil movement profile was investigated, and all piles in a group are connected with a pile cap. The results obtained are shown to be consistent, with a number of general pile behaviour trends identified. Table 5.12 tabulates these trends, addressing the following themes, according to each pile group arrangement:

1. the effect of the pile location, in a group, on the pile bending moment
2. the magnitude of $w_s$ in order to mobilise the peak $M_{\text{max}}$ and $r_{\text{max}}$
3. the shape of the bending moment profile
4. the effect of the axial load on the pile response
5. the effect of the $L_m/L_s$ ratio
6. the effect of the pile diameter
7. the effect of the pile spacing
8. limiting pressure profile
9. the $M_{\text{max}}$ against $S_{\text{max}}$ relationship
10. other significant pile behaviours.
Table 5.12 General pile behaviour trends identified in the pile group tests

<table>
<thead>
<tr>
<th>Theme</th>
<th>1 × 2 pile group (5 tests)</th>
<th>2 × 1 pile group (4 tests)</th>
<th>2 × 2 pile group (12 tests)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>The bending moments of piles A and B were almost identical.</td>
<td>With L_m/L_s = 200/500, the M_max on pile A was lower than that on pile B. While M_max on pile A was higher than on pile B. With L_m/L_s = 400/300, generally, the M_max on pile A was lower than that on pile B.</td>
<td>With L_m/L_s = 200/500, M_max and M_max on pile A were generally higher than on pile B. With L_m/L_s = 400/300, M_max on pile B was slightly higher than pile A.</td>
</tr>
<tr>
<td>2</td>
<td>With L_m/L_s = 200/500, the M_max and the r_max, reached their peak at w_s = 30 mm.</td>
<td>The pile response (in terms of M_max and the r_max) reached its peak and remained virtually constant at w_s ≥ 40 mm and w_s ≥ 60 mm for tests with L_m/L_s of 200/500 and 400/300, respectively.</td>
<td>With L_m/L_s = 200/500, the M_max and the r_max, reached its peak and tended to remain unchanged at w_s ≥ 40 mm. With L_m/L_s = 400/300, the M_max and the r_max, continued to increase as w_s increased, without reaching a definite peak.</td>
</tr>
<tr>
<td>3</td>
<td>With L_m/L_s = 200/500, the shape of the bending moment profiles showed a single curvature (on piles A and B).</td>
<td>The shape of the bending moment profiles showed a double curvature (on piles A and B).</td>
<td>The shape of the bending moment profiles showed a double curvature (on piles A and B).</td>
</tr>
<tr>
<td></td>
<td>With L_m/L_s = 400/300, the shape of the bending moment profiles showed a double curvature (on piles A and B).</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>The axial load increased with M_max.</td>
<td>With L_m/L_s = 200/500, additional the axial load applied on the pile group increased the M_max and M_max on pile A, but on pile B no significant increase in M_max was noted.</td>
<td>With L_m/L_s = 200/500, axial load increased the M_max and the M_max on pile A, where as on pile B, the axial load increased the M_max, but reduced the M_max. With L_m/L_s = 400/300, axial load increased the M_max on piles A and B.</td>
</tr>
<tr>
<td>5</td>
<td>M_max on piles A and B increased with the L_m/L_s ratio.</td>
<td>M_max on piles A and B increased with L_m/L_s ratio.</td>
<td>M_max increased with L_m/L_s ratio (on piles A and B).</td>
</tr>
<tr>
<td>6</td>
<td>N/A</td>
<td>N/A</td>
<td>The M_max and M_max increased with the pile diameter (on piles A and B).</td>
</tr>
</tbody>
</table>
Chapter 5: Pile group tests

<table>
<thead>
<tr>
<th>Theme</th>
<th>1 × 2 pile group (5 tests)</th>
<th>2 × 1 pile group (4 tests)</th>
<th>2 × 2 pile group (12 tests)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>The pile spacing had a minimum effect on the $M_{\text{max}}$</td>
<td>N/A</td>
<td>With $L_m/L_s = 200/500$, an increase in the pile spacing increased $M_{\text{max}}$ and $M_{\text{max}}$, on pile A, while on pile B, only $M_{\text{max}}$ increased with the pile spacing (however for a larger pile diameter, the increments tended to reduce). With $L_m/L_s = 400/300$, an increase in the pile spacing increased the $M_{\text{max}}$, but reduced $M_{\text{max}}$ (significantly), on pile A.</td>
</tr>
<tr>
<td>8</td>
<td>In $L_m$, the $p_{\text{max}}$ generally did not exceed the limiting pressure proposed by Barton (1982) and Broms (1964) on the tests with $L_m/L_s = 200/500$ and 400/300, respectively.</td>
<td>In $L_m$, on the tests with $L_m/L_s = 200/500$ and 400/300, the $p_{\text{max}}$ did not exceed the limiting pressure proposed by Broms (1964) and Barton (1982), respectively.</td>
<td>In $L_m$, on the tests with $L_m/L_s = 200/500$, the $p_{\text{max}}$ on pile A were significantly higher than on pile B. The magnitude of the $p_{\text{max}}$ on piles A and B did not exceed the limiting pressure proposed by Barton (1982) and Broms (1964), respectively.</td>
</tr>
<tr>
<td></td>
<td>In $L_s$, the $p_{\text{max}}$ generally did not exceed the limiting pressure proposed by Broms (1964) and Barton (1982) on the tests with $L_m/L_s = 200/500$ and 400/300, respectively.</td>
<td>In $L_s$, the $p_{\text{max}}$ generally did not exceed Rankine’s passive pressure and the limiting pressure proposed by Barton (1982) on the tests with $L_m/L_s = 200/500$ and 400/300, respectively.</td>
<td>In $L_s$, on the tests with $L_m/L_s = 200/500$, the $p_{\text{max}}$ on piles A and B did not exceed Rankine’s passive pressure.</td>
</tr>
<tr>
<td>9</td>
<td>Tests results fitted $M_{\text{max}} = 0.24S_{\text{max}}$</td>
<td>Tests results fitted $M_{\text{max}} = 0.22S_{\text{max}}$</td>
<td>Tests results fitted $M_{\text{max}} = 0.23S_{\text{max}}$</td>
</tr>
<tr>
<td>10</td>
<td>At $L_m/L_s = 200/500$, the bending moment profiles on piles A and B, respectively, were similar in terms of shape and magnitude to that on the single pile.</td>
<td>At $L_m/L_s = 200/500$, the $M_{\text{max}}$ recorded in the pile group were always lower than the single pile for the same $L_m/L_s$ ratio.</td>
<td>At $L_m/L_s = 200/500$, a unique “shadowing” effect was observed in the pile group. This effect caused the soil pressure acting on pile A (front row) to be significant higher than that on pile B (back row). At $L_m/L_s = 400/300$ the “shadowing” effect was not noted on piles A and B, as both piles behaved in a similar manner.</td>
</tr>
</tbody>
</table>
Chapter 6: Three-Dimensional Finite Difference Analysis

6.0 Introduction

A series of numerical analyses were performed to predict the results of the model tests described in Chapter 4. Previous work, by Chen (1994), Chen (2002) and Lee (2004), had focused mainly on piles subjected to triangular profile of lateral soil movements in sand. Due to the complexity of the problem where the piles are subjected to both axial load and lateral soil movement, simultaneously, a complete analysis is only possible with the help of a three-dimensional numerical analysis. In this chapter, the single piles tests, subjected to axial load and rectangular soil movement profile were analysed. Additional analyses, that were performed to study the effect of a larger range of \( L_m/L_s \) ratios and the dilation angle of sand on the pile behaviour, which had not been previously investigated in an experimental study, are discussed.

6.1 Program details

FLAC\textsuperscript{3D} (Fast Lagrangian Analysis of Continua in 3 Dimensions), a finite difference program, was used in the numerical analysis. Eleven constitutive models are available in the FLAC\textsuperscript{3D} version 2.1. These models were most commonly used in modelling geotechnical problems. More details on the capabilities of the program can be found in the program manual (Itasca, 2002).

One of the main features of the FLAC\textsuperscript{3D} is that it can operate in a small or large strain mode. The large strain modes occur where the grid point (mesh) coordinates are updated at each strain or movement, according to the computed displacement. This mode is particularly important to the present study due to the nature of the problem, in which large soil movements is involved.

6.2 Standard model setup

A standard model was established as a bench mark for the later parametric study, and for the common input parameters used in the detailed analyses of the model tests.
6.2.1 Geometry and mesh

In order to capture the effect of soil arching, pile diameter and soil flowing, the three-dimensional model was most effective. Taking the advantage of the symmetrical nature of the model; thus only half of the shear box was modelled, as shown in Figure 6.1. The soil strata were modelled with eight-noded brick shaped elements, while the pile was modelled with six-noded cylindrical shaped elements. The interface elements were placed between the soil and the pile. For the standard model, with a single pile, the mesh comprised of 1856 elements with 2894 nodes or grid-points. While a greater accuracy can be achieved by using a higher number of elements to model the soil and the pile, this also leads to a denser mesh. However, as detailed in the later section, the use of the denser mesh only leads to a slight increase in the accuracy of the result, but requires a longer solution time; hence this option was not implemented for the present study. With the current computer capacity (Pentium®4 processor), approximately two to six days were required to analyse a standard model.

The bottom face of the mesh (in Figure 6.1) was fixed in all three directions, x, y and z. The surface of the mesh was not fixed in any direction. Both faces parallel to the yz-plane were fixed in the x direction; the other faces parallel to the xz-plane were fixed in the y direction. The surface of the mesh was not fixed in any direction.

![Figure 6.1 Mesh of soil and pile used in FLAC\textsuperscript{3D} analysis](image)
Chapter 6: *Three-dimensional finite difference analysis*

6.2.2 Sand properties

The sand in the moving \((L_m)\) and the stable \((L_s)\) layers was assigned a friction angle of \(\phi = 38^\circ\) (taken as the peak friction angle obtained from the direct shear test) and Poisson’s ratio of 0.3. The dilation angle, \(\psi\), was estimated according to the equation: \(\psi = \phi - 30^\circ\) (after Chae et al, 2004). Hence, the \(\psi\) for the dense sand used in the present study, using \(\phi = 38^\circ\), was 8°, which was within the range of \(\psi = 0^\circ\) to 20° for the sands reported by Vermeer and deBorst (1984), and Bolton (1986). The Young’s modulus of sand \(E_s\) was assigned as 572 kPa on both the \(L_m\) and the \(L_s\) layers. This value was obtained from the oedometer test data on the sand (calculated from constrained modulus and \(v_s = 0.3\)). Both the direct shear and the oedometer test details can be found in Appendix A. The initial stress was calculated by specifying a sand density of 16.27 kN/m\(^3\) and a coefficient of earth pressure at rest, \(K_o = 1 - \sin \phi\) (Jaky, 1944). The sand was assumed to have zero tensile strength \((\sigma_t = 0)\).

6.2.3 Mohr-Coulomb constitutive model for sand

The sand strata were modelled with elasto-plastic Mohr-Coulomb constitutive model, together with non-associated flow rule. The Mohr-Coulomb failure criterion was expected to contribute an important part at the moving layer where the large deformations of the sand were expected. Furthermore, the Mohr-Coulomb model, widely used in modelling geotechnical problem, is relatively simpler to use when compared to other constitutive models. Except for Subsection 6.4.2, all the analyses presented in this chapter were carried out with the Mohr-Coulomb model.

6.2.4 Interface model

In most soil-structure interaction problems, relative movement of the soil, with respect to the structure (pile), can occur. This movement was noted in the model pile tests reported in Chapter 4, and from the photographs taken during the experiment (see Chapter 3, Figure 3.7). Most of the early developments of the three-dimensional numerical analysis used the continuum elements with a compatibility of displacement, and hence they restrict relative movement at the soil-pile interface. In order to
overcome this limitation, FLAC\textsuperscript{3D} provided an interface which allows differential movement between the soil and the pile. This capability enables slip and separation to occur. Further, the interface characterised by Coulomb sliding and tensile separation, had the properties of friction dilation, cohesion, normal and shear stiffnesses, and tensile strength (Itesca, 2002). The Coulomb theory applies when two surfaces are in contact. The contact forces, at the interface (soil-pile), are transferred only at the interface nodes. However, when the contact force reaches its maximum tensile and shear stresses, a separation and slip occurs between the two surfaces, respectively. Itasca (2002) recommended that the lowest stiffness, consistent with a small interface deformation, be used. Thus, \( k_n \) were selected and so there was no significant penetration, while \( k_s \) were selected, with no noticeable slip displacement occurring high enough between the pile and the sand.

In the current study, an interface model was used to model the connection between the soil and the pile. This interface model was defined by Coulomb’s failure criterion, had the following parameters: 1) friction angle, \( \phi_i = 28^\circ \) (interface between sand and pile, taken as less than the friction angle of sand); 2) normal stiffness (\( k_n \)) and shear stiffness (\( k_s \)) of \( 1.0 \times 10^8 \) N/m\(^2\)/m; 3) cohesion (\( c_i \)) and dilation value (\( \psi_i \)) of zero; and 4) tensile strength, \( \sigma_t = 0 \). As previously mentioned, the interface element allows for a separation between the soil and the pile, hence there was no tensile force acts on the pile (with \( \sigma_t = 0 \)) in an event when the soil was moving away from the pile. In most analyses, this event occurs at the surface of the soil as the magnitude of the soil movement increased.

6.2.5 Model pile

In all the loading conditions of the model tests presented in Chapter 4, the measured bending moments on the piles were well below the yield moment. Therefore, the piles used in the model tests were represented as an isotropic elastic hollow pile. The internal and the external diameters of the isotropic elastic hollow piles were the same as for the piles used in the model tests. As noted earlier, due to the symmetrical nature of the problem, only half of the piles were modelled.
6.2.5.1 Calibration of model piles

The piles were calibrated by modelling a free standing pile fixed in all directions at the bottom (simple cantilever setup as shown in Figure 6.2(a)). The length of the pile was 1.2 m, and had a hollow cross-section, with the internal and the external diameters of 46.2 mm and 50.0 mm respectively, thus reflecting the actual dimensions of the pile used in model tests. The properties of the pile had the Young’s modulus of aluminium, \( E_p = 7.0 \times 10^{10} \text{ N/m}^2 \), and Poisson’s ratio of \( \nu_p = 0.33 \). A lateral load of 300 N was applied on the top of the pile. The deflection and bending moment along the pile was calculated using simple static equations (Timoshenko and Young, 1945), and were compared to the results obtained from the FLAC\(^{3D}\) analysis, as shown in Figures 6.2(b) and (c), respectively. The good fit comparison shows that FLAC\(^{3D}\) was sufficiently accurate to model the pile as an elastic member.

Figure 6.2 Cantilever model pile calculated with static equations and FLAC\(^{3D}\)
6.2.6 Summary of the input parameters for the standard model

Table 6.1 shows a summary of the input parameters for the standard model. These parameters were used in all FLAC$^3$D analyses performed in this chapter, unless stated otherwise. The FLAC$^3$D models were built to the exact dimensions of the model tests. Due to time limitations, only single pile tests were model in FLAC$^3$D. Generally, the duration of an analysis, using a Pentium 4 processor, is approximately two days. However, for the model tests, with an axial load on the pile head, FLAC$^3$D analyses require six days to complete.

Table 6.1 Input parameters for the standard model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand:</td>
<td></td>
</tr>
<tr>
<td>Soil Young’s modulus, $E_s$</td>
<td>572 kPa</td>
</tr>
<tr>
<td>Friction angle, $\phi$</td>
<td>$38^\circ$</td>
</tr>
<tr>
<td>Dilation angle, $\psi = \phi - 30^\circ$</td>
<td>$8^\circ$</td>
</tr>
<tr>
<td>Poisson ratio, $\nu_s$</td>
<td>0.30</td>
</tr>
<tr>
<td>Constitutive model</td>
<td>Mohr-Coulomb</td>
</tr>
<tr>
<td>Pile:</td>
<td></td>
</tr>
<tr>
<td>Pile Young’s modulus, $E_p$</td>
<td>70 GPa</td>
</tr>
<tr>
<td>Poisson ratio, $\nu_p$</td>
<td>0.33</td>
</tr>
<tr>
<td>Pile diameter</td>
<td>50 mm</td>
</tr>
<tr>
<td>Constitutive model</td>
<td>Elastic</td>
</tr>
<tr>
<td>Interface element between pile and soil:</td>
<td></td>
</tr>
<tr>
<td>Interface properties</td>
<td></td>
</tr>
<tr>
<td>a) shear stiffness, $k_s$</td>
<td>$1 \times 10^8$ N/m$^2$/m</td>
</tr>
<tr>
<td>b) normal stiffness, $k_n$</td>
<td>$1 \times 10^8$ N/m$^2$/m</td>
</tr>
<tr>
<td>c) friction angle, $\phi_i$</td>
<td>$28^\circ$</td>
</tr>
<tr>
<td>Interface model</td>
<td>Mohr-Coulomb</td>
</tr>
</tbody>
</table>

6.3 Application of stress and movements

In order to model piles subjected to lateral soil movements, a lateral velocity was applied at the direction parallel to the x-axis, as shown in Figure 6.1(a). Previous experience gained from calibrating the model pile indicated that the minimum ‘steps’ required for the “unbalanced force”, to arrive at a small and constant value, without significant changes, was approximately 150,000 steps. Two criteria exist in selecting the right velocity (steps per 1 mm of lateral soil movement):

- the velocity should be as high as possible to minimise the truncation errors that arise when too many significant figures are lost, when especially very
small displacement increments are added to coordinate the values in a large strain mode (Coulthard, 2005); and

- the velocity should be as low as possible to achieve a small value of the “unbalanced force”, which is also an indication of the degree of the convergence of the model.

After a few trials, it was found that the level of axial load, applied to the pile, had an influence on the optimum velocity needed for the FLAC3D model to converge (to achieve minimum unbalanced force). Consequently, the velocity adopted varied from 10,000 steps/mm to 100,000 steps/mm for the models with Q = 0 N and Q = 294 N, respectively.

6.4 Comparative study on the standard model

A comparative study of the standard model was undertaken to assess the following:

1) Effect of mesh density, Subsection 6.4.1
2) Effect of constitutive model, Subsection 6.4.2.

It should be noted that the standard model had the Lm/Ls ratio of 400/300.

6.4.1 Effect of mesh density

While the parametric studies had been conducted using the standard mesh (Figure 6.1), it was of interest to investigate the sensitivity and the efficiency of the mesh in terms of the solution time and file size. Two alternative meshes (1 and 2), with a higher number of zones, were created to study the effect of the mesh density on the pile response. The meshes details are:

1) Standard mesh, in Figure 6.1: 1856 elements (or zones) and 2894 grid points
2) Mesh 1, in Figure 6.3(a): 2976 elements (or zones) and 4406 grid points
3) Mesh 2, in Figure 6.3(b): 2368 elements (or zones) and 3488 grid points.

Figures 6.3(c) and (d) show the pile response obtained from the three mesh densities. The differences in the $M_{\text{max}}$ and the maximum pile deflections obtained between the different meshes ranged from 15 % to 3 %, respectively. It is important to note that meshes 1 and 2 had significantly more elements when compared to the standard mesh.

The above comparison shows that the standard mesh requires less solution time and
generates a smaller file size, while it has a similar degree of accuracy to that of the denser meshes. For this reason, the standard mesh was adopted for all later models.

Figure 6.3 The effect of mesh density on the pile response

a) Model with increased horizontal grids (2976 elements)
b) Model with increased vertical grids (2368 elements)

(c) (d)
6.4.2 Effect of constitutive models

A comparison was undertaken of the bending moment and pile deflection, between the two constitutive models, namely Mohr-Coulomb and Drucker-Prager (Figures 6.4(a) and (b)). These models were used to model the behaviour of the dry sand in the shear box. Generally the Mohr-Coulomb and Drucker-Prager constitutive models give two similar bending moment profiles, except at the depth to 200 mm. At that depth, the bending moment obtained from the Mohr-Coulomb model is 127 % higher than the bending moment obtained from the Drucker-Prager model. This difference can be attributed to the differing plastic strength limits of each model, since above the sliding surface (in $L_m$), the majority of the sand elements are in a plastic state. Below the sliding surface (in $L_s$) of 400 mm, the bending moments obtained from the constitutive models are similar, with only 1 % difference. This response is expected as the majority of the sand below this depth is in an elastic state (both models have the same response in the elastic range). The Mohr-Coulomb model is adopted in the future analysis as it predicted well, the behaviour of the sand (Itasca, 2002; Vermeer and deBorst, 1981).

![Figure 6.4 Comparison between Mohr-Coulomb and Drucker-Prager constitutive models](image)

Figure 6.4 Comparison between Mohr-Coulomb and Drucker-Prager constitutive models
6.4.3 Parametric study

A parametric study was conducted, in order to investigate the sensitivity of each of the sand parameters on the standard FLAC\textsuperscript{3D} model (Table 6.1). One of these parameters was varied at a time, to enable a comparison with the results from the standard model. The varied parameters varied are summarised in Table 6.2, with the results being presented, mainly, in terms of the bending moment and the pile deflection profiles, at \( w_s = 60 \text{ mm} \).

<table>
<thead>
<tr>
<th>Varied parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Soil Young's modulus:</strong></td>
<td></td>
</tr>
<tr>
<td>a) ( 0.5 \times E_s )</td>
<td>286 kPa</td>
</tr>
<tr>
<td>b) ( 2 \times E_s )</td>
<td>1,144 kPa</td>
</tr>
<tr>
<td>c) ( 10 \times E_s )</td>
<td>5,720 kPa</td>
</tr>
<tr>
<td>d) ( E_s ) varies with depth (linearly)</td>
<td>334 kPa at 0mm depth to 744 kPa at 800mm depth</td>
</tr>
<tr>
<td><strong>Dilation angle:</strong></td>
<td></td>
</tr>
<tr>
<td>a) ( \psi (= \phi, \text{associate flow rule}) )</td>
<td>38°</td>
</tr>
<tr>
<td>b) ( \psi ) (average calculated from direct shear test)</td>
<td>11.5°</td>
</tr>
<tr>
<td>c) ( \psi ) (for non dilating sand)</td>
<td>0°</td>
</tr>
<tr>
<td><strong>Interface element between pile and soil:</strong></td>
<td>Soil fully attached to the pile</td>
</tr>
<tr>
<td>a) without interface element</td>
<td></td>
</tr>
<tr>
<td>b) ( \phi )</td>
<td>38°</td>
</tr>
<tr>
<td><strong>Interface strength</strong></td>
<td></td>
</tr>
<tr>
<td>c) ( 0.1k_n ) and ( 0.1k_s )</td>
<td>( 1 \times 10^7 \text{ N/m}^2/\text{m} )</td>
</tr>
<tr>
<td>d) ( 0.01k_n ) and ( 0.01k_s )</td>
<td>( 1 \times 10^6 \text{ N/m}^2/\text{m} )</td>
</tr>
</tbody>
</table>

*Note: For comparison, the soil properties for standard model are found in Table 6.1*
6.4.3.1 Effect of Young’s modulus

The sensitivity of the Young’s modulus of the sand on the FLAC$^3$D model (Mohr-Coulomb) was investigated using four values, as shown in Table 6.2. In the first three cases, each $E_s$ (286 kPa, 1,114 kPa and 5,270 kPa) was set to be constant along the entire depth of the shear box; and the last case, was varied linearly with a depth from 334 kPa at the sand surface to 744 kPa at a depth of 800 mm. The bending moment and pile deflection profiles for all four cases are plotted in Figure 6.5. Figure 6.5(a) shows that, by increasing the $E_s$ of the standard model to 10 times, the bending moments along the pile increases most notably at a depth of 500 mm. As expected, when the pile is embedded in a stiffer soil ($10 \times E_s$), the deflection of the pile reduces to approximately 14.5 % from that obtained from the least stiff soil ($0.5 \times E_s$). In between, the sand with $E_s$ (standard model) and $E_s$ varies with depth, the differences in the positive maximum bending moment ($M_{+\text{max}}$) and the maximum pile deflection are 17.5 % and 3.3 %, respectively. Given these small differences (< 20 %), all models described in this chapter (unless specified otherwise) have been analysed with $E_s$, taken as constant with depth, to save computational time.

Figure 6.5 The effect of Young’s modulus on the pile response
6.4.3.2 Effect of dilation angle of sand

Three additional dilation angles of $\psi = 0^\circ$, 11.5° and 38° were investigated for the effect of the dilation angle of sand on the FLAC$^{3D}$ model. The computed results, using the standard model with different dilation angles of sand, are shown in Figure 6.6. As shown in Figure 6.6(a), an increase in $\psi$, from 8° to 38°, leads to an increase of 56 % and 8 % on the negative and positive bending moment, respectively. The $\psi = 8^\circ$ is currently adopted in the standard model. However, should the $\psi = 11.5^\circ$ (average measured from the direct shear box test) be adopted instead, the differences in $M_{\text{max}}$ and pile deflection, respectively, are minor (in the range of < 2 %). In general, the different pile responses show that, as the sand dilation angle increases, the soil pressure at a given $w_s$ increases; this, in turn, increases the load acting on the pile (from the moving soil) and the bending moment. In this case, where $L_m/L_s$ is 400/300, the dilation angle has little influence on the $M_{\text{max}}$ and maximum pile deflection; however, it has significant influence on the $M_{\text{max}}$. Indeed, the dilation angle of sand has a greater influence on the bending moment profile when the sliding depth is shallow ($L_m < L_s$). This effect will be discussed later in the chapter.

![Figure 6.6 The effect of dilation angle of sand on the pile response](image_url)
6.4.3.3 Effect of interface element

*Interface strength*

A total of three analyses, as presented in Figures 6.7(a) and (b) were undertaken: one analysis with the pile elements fully attached to the soil elements, hence no interface elements were provided; and two analyses with the pile and the soil elements were separated by the interface elements. In the analyses with the interface elements, two different interface friction angles ($\phi_i$) were assigned to the interface elements namely: 1) $\phi_i = 28^\circ$; and 2) $\phi_i = 38^\circ$. These analyses had $k_n$ and $k_s$ assigned, respectively, equal to $1.0 \times 10^8$ N/m$^2$/m. The comparisons, in term of the bending moment and the pile deflection, respectively, show very similar responses between the three analyses. The differences are in the range of 3.0 % and 4.7 % for the $M_{r_{\text{max}}}$ and the pile deflection (at the surface), respectively. Therefore, the interface element used in the standard model only has a minor effect on the pile response.

*Interface stiffness*

A total of four analyses were undertaken in the study (Figures 6.7(c) and (d)): one analysis with pile elements fully attached to the soil, hence no interface elements were provided; and three analyses with the pile and the soil elements separated by the interface elements. In the analysis with interface elements, all the interfaces had the same strength properties ($\phi_i = 28^\circ$), but with stiffness properties ($k_n$ and $k_s$) assigned as: 1) $k_n$ & $k_s = 1 \times 10^8$ N/m$^2$/m; 2) $k_n$ & $k_s = 1 \times 10^7$ N/m$^2$/m (labelled as 0.1$k_n$ & 0.1$k_s$ in Figures 6.7(c) and (d)); 3) $k_n$ & $k_s = 1 \times 10^6$ N/m$^2$/m (labelled as 0.01$k_n$ & 0.01$k_s$ in Figures 6.7(c) and (d)). The comparisons, in terms of $M_{r_{\text{max}}}$ and pile deflection, respectively, shows that the analysis with $k_n$ & $k_s = 1 \times 10^8$ N/m$^2$/m is closest to the analysis results without the interface elements. As the interface properties increase from $1 \times 10^6$ N/m$^2$/m to $1 \times 10^8$ N/m$^2$/m, the shape of the bending moment profile changes and becomes closer to, or resembles more, the analysis without the interface elements.
When the interface elements were used, the $k_n$ and $k_s$ properties were set to a value high enough to prevent any excessive penetration or slip at the interface. The main purpose of the interface elements was to allow the pile and the soil to separate when the soil was moving away from the pile (mainly noticeable at the surface of the soil). Other movements such as the slip and penetration at the interface were thus minimized. Ideally, any excessive movement around the pile should be result of the yielding of the soil elements (piles are unlikely to yield) rather than the interface elements. However, care needed to be taken not to set an excessively high value of the interface stiffness properties, which could cause a significant increase in the computational time.

Figure 6.7 The effect of interface elements on the pile response: (a)-(b) with different interface strengths; and (c)-(d) with different interface stiffness
6.4.4 Summary of the parametric study

In summary, the parametric study shows that the Young’s modulus and the dilation angle of the sand have the most influence on the pile response (Table 6.3). The value of Young’s modulus is obtained from the Oedometer tests. The dilation angle, at this stage, is based on the estimated values of the published results. In order to compare the FLAC$^{3D}$ model with the experimental model tests, the effect of the dilation angle is studied, and the findings are presented in detail later.

Table 6.3 Summary of the results obtained from the parametric study

<table>
<thead>
<tr>
<th>Comparison</th>
<th>% difference from the standard model</th>
<th>Figure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$M_{max}$</td>
<td>$M_{max}$</td>
</tr>
<tr>
<td>Mesh density:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mesh 1</td>
<td>15</td>
<td>9</td>
</tr>
<tr>
<td>Mesh 2</td>
<td>3</td>
<td>14</td>
</tr>
<tr>
<td>Constitutive model:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Drucker-Prager</td>
<td>1</td>
<td>127</td>
</tr>
<tr>
<td>Young’s modulus:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$0.5 \times E_s$</td>
<td>31</td>
<td>9</td>
</tr>
<tr>
<td>$2 \times E_s$</td>
<td>56</td>
<td>32</td>
</tr>
<tr>
<td>$10 \times E_s$</td>
<td>220</td>
<td>n/a</td>
</tr>
<tr>
<td>$E_s$ varies with depth</td>
<td>18</td>
<td>8</td>
</tr>
<tr>
<td>Dilation angle:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\psi = 0^\circ$</td>
<td>11</td>
<td>32</td>
</tr>
<tr>
<td>$\psi = 11.5^\circ$</td>
<td>11</td>
<td>0</td>
</tr>
<tr>
<td>$\psi = 38^\circ$</td>
<td>8</td>
<td>56</td>
</tr>
<tr>
<td>Interface element between pile and soil:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>a) without interface element</td>
<td>2</td>
<td>14</td>
</tr>
<tr>
<td>Interface strength</td>
<td></td>
<td></td>
</tr>
<tr>
<td>b) $\phi = 38^\circ$</td>
<td>3</td>
<td>9</td>
</tr>
<tr>
<td>Interface stiffness</td>
<td></td>
<td></td>
</tr>
<tr>
<td>c) $k_n = k_s = 1 \times 10^7$ N/m$^2$/m</td>
<td>2</td>
<td>150</td>
</tr>
<tr>
<td>d) $k_n = k_s = 1 \times 10^6$ N/m$^2$/m</td>
<td>n/a</td>
<td>16</td>
</tr>
</tbody>
</table>
6.5 FLAC\(^{3D}\) analyses performed on model tests

A large number of model tests were conducted; owing to the time constraint, only five model test results were analysed, in order to: 1) verify the model test results on the different levels of the axial force applied on the piles; and 2) examine the effect of \(L_m/L_s\) ratio and the pile stiffness. These FLAC\(^{3D}\) analyses are outlined in Table 6.4.

Table 6.4 FLAC\(^{3D}\) analyses performed on model tests

<table>
<thead>
<tr>
<th>Pile diameter (mm)</th>
<th>Axial load, (Q) (N)</th>
<th>Moving layer, (L_m) (mm)</th>
<th>Stable layer, (L_s) (mm)</th>
<th>FLAC(^{3D}) model reference</th>
<th>Figure</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>0</td>
<td>400</td>
<td>300</td>
<td>F_4-3_50_0k_600</td>
<td>6.9</td>
</tr>
<tr>
<td>50</td>
<td>294</td>
<td>400</td>
<td>300</td>
<td>F_4-3_50_30k_600</td>
<td>6.10</td>
</tr>
<tr>
<td>50</td>
<td>589</td>
<td>400</td>
<td>300</td>
<td>F_4-3_50_60k_600</td>
<td>6.11</td>
</tr>
<tr>
<td>50</td>
<td>0</td>
<td>200</td>
<td>500</td>
<td>F_2-5_32_0k_600</td>
<td>6.13</td>
</tr>
<tr>
<td>32</td>
<td>0</td>
<td>200</td>
<td>500</td>
<td>F_2-5_50_0k_600</td>
<td>6.14</td>
</tr>
</tbody>
</table>

In the following subsections, the results from the FLAC\(^{3D}\) analyses are presented together with the results from the model tests, in terms of the five profiles, namely: 1) bending moment; 2) shear force; 3) soil reaction; 4) pile rotation; and 5) pile deflection. These profiles are obtained at the \(w_s = 60\) mm.
6.5.1 Test on 50 mm pile at $L_m/L_s = 400/300$ and $Q = 0$ N

The FLAC$^{3D}$ analyse was performed to compare with the results obtained from the respective model test (see Table 4.1, test 4-3_50_0k_600_100904). The input parameters used in the FLAC$^{3D}$ model can be seen in Tables 6.1 and 6.4. Figure 6.8 shows the five profiles of the pile. The differences in the $M_{\text{max}}$ and $M_{\text{max}}$, between the FLAC$^{3D}$ and the model test, are 13% and 151%, respectively. Although the magnitude of the bending moment profiles show greater differences, they are similar in shape, with both having double curvatures (negative and positive bending moments). Generally the shear force, soil reaction, rotation and pile deflection profiles show the FLAC$^{3D}$ under the predicted response for the entire length of the pile. For example, the pile deflection from the model tests at the soil surface is 24.2 mm, and is higher than that from the FLAC$^{3D}$.

Figure 6.9 shows the magnitude of the soil and pile displacement contours in the shear box at intervals of 30 mm, 60 mm, 90 mm and 120 mm of the applied $w_s$ ($w_s$ is taken as the applied displacement at the boundary of the shear box, see Figure 6.1(a)). At $w_s = 30$ mm, the soil displacement contour shows a triangular wedge within the shear box. As $w_s$ increases to 120 mm, the shape of the wedge changes to a more uniform soil movement contour across the shear box. There is a distinctive “cut-off” at the interface between the $L_m$ and the $L_s$ layers.

Figure 6.10 shows the magnitude of the stress (x-direction) build up in the shear box at different $w_s$, with the stress development indentified as:

1) At $w_s = 30$ mm, the stress building up is most noticeable at $L_m$

2) At $w_s = 60$ mm, the stress continues to build up, an extension of the stress concentration is in the lower part of $L_m$. The section for the pile subjected to stress in the $L_m$ and $L_s$ are almost equal in length. At the vicinity of the sliding depth, the stress on the pile changes in direction, indicating a reaction provided by the pile to resist the sliding soil, entirely from the $L_s$ layer

3) At $w_s = 90$ mm, the stress contours continues to build up, with the maximum stress noticeable in the $L_s$
4) At $w_s = 120$ mm, the pile section in $L_s$ is a rigid pile with the point of rotation closer to the pile tip. Also, at all levels of $w_s$, the top 100 mm depth of the pile shows no noticeable stresses build up.
Figure 6.8 Pile responses of Test F_4-3_50_0k_600
Figure 6.9 Contour showing the magnitude of soil and pile displacement
Figure 6.10 The magnitude of stress (x-direction) build up in the shear box
6.5.2 Test on 50 mm pile at $L_m/L_s = 400/300$ and $Q = 294$ N

The response of 50 mm diameter model pile, with $L_m/L_s = 400/300$ and $Q$ of 294 N, applied on the pile head at 500 mm above the soil surface, is presented in Figure 6.11. The comparison shows that similar bending moment profiles (shape) are obtained from both the model test and the FLAC$^{3D}$ analysis. This similarity is also noted on the shear force, the soil reaction and the pile rotation profiles, but not the pile deflection profile. The $M_{\text{max}}$ obtained from the FLAC$^{3D}$ analysis is 49 % higher than that obtained from the model test. Generally, the results from the FLAC$^{3D}$ are higher, in terms of magnitude, than that from the model test. The differences in the maximum shear forces and the maximum soil reactions are 55 % and 70 %, respectively (Figures 6.11(b) and (c)). As shown in Figure 6.11(d), the rotation angles for both the model test and the FLAC$^{3D}$ are -0.038 and -0.085, respectively. The pile deflection obtained from the FLAC$^{3D}$ at the soil surface is 61.18 mm, as compared with the model test results of 59.52 mm, hence, giving a difference of only 3 %. At the pile toe, the pile deflections obtained from the model test and FLAC$^{3D}$ are -32.77 mm and -7.635 mm, respectively. As indicated in Figure 6.11(e), the pile from the FLAC$^{3D}$ only rotates as the soil movement increases. On the other hand, as discussed in Chapter 4, the model test shows that the pile first rotates and later translates, as the soil movement increases.

As reported in Chapter 4 (Subsection 4.2.3.4), the pile is not perfectly vertical before the soil movement is applied, with the following effects on the pile response from the model test:

1) the initial pile deflection and pile rotation are not zero (negative value), due to a slight deviation of the pile from a perfectly vertical position. As a result, the pile deflection and the pile rotation have a lower net value (positive value) as the soil movement increases

2) a reduction in the “p-delta” (axial load multiply by pile deflection) effect on the pile due to a lower “delta” after the initial negative deflection of the pile is offset (see Figure 6.13).

It is not surprisingly that the FLAC$^{3D}$ analysis gives higher values on all the pile responses than that from the model tests.
Figure 6.11 Pile responses of Test F_4-3_50_30k_600
6.5.3 Test on 50 mm pile at L_m/L_s = 400/300 and Q = 589 N

The response of the 50 mm diameter pile with L_m/L_s = 400/300 and Q = 589 N, is shown in Figure 6.12. In the model test, the load was applied by means of placing weight blocks, with a total of 589 N, on the pile head located 500 mm above the soil surface, as shown in Figure 6.13. In the FLAC\textsuperscript{3D} analysis, this load was applied as an equivalent uniform pressure, distributed on the pile elements at the pile head level. The pile response from both the model test and the FLAC\textsuperscript{3D} are similar in terms of their profiles. However, the M_{max} obtained from the model test is 67.7 % higher than that obtained from the FLAC\textsuperscript{3D}. Furthermore, both the shear force and the soil reaction are obtained from single and double differentiations of the bending moment, respectively, and so these responses from the model test are higher in magnitude as compare to that from FLAC\textsuperscript{3D} (Figures 6.12(b) and (c)).

As previously shown in Subsection 6.5.1, when the pile is subjected to only lateral soil movement, the location of the maximum shear force (S_{max}) is in the vicinity of the depth of the moving layer L_m, and the location of the M_{max} is located within the stable layer L_s. These locations (S_{max} and M_{max}) are also found to be the case for the pile with Q = 294 N (Subsection 6.5.2).

One noticeable effect of the axial load, applied on the pile head, is the positive bending moment induced at the soil surface, as a result of the p-delta effect, as the pile deflected. The difference in the bending moments (at the sand surface) between the model test and the FLAC\textsuperscript{3D} is 127 %, and is also attributed to the way the load is applied on the pile head (Figure 6.13). In this model test, where the pile is perfectly vertical, prior to the application of soil movement, the force distribution (or stress, weight over the contact area) for both the model test and the FLAC\textsuperscript{3D} analyses are the same. In the model test, as the soil movement increases, the pile begins to deflect. As the pile deflects, the connector block placed (without any bolt-fasten connection) on the pile head creates a non-uniform distribution of force on the pile head. On the other hand, in the FLAC\textsuperscript{3D} analysis the force is uniformly applied and remains unchanged at all levels of the soil movement. As a result of this difference in the force distribution, the p-delta effect on the model test (a larger force concentration off the centreline of
the pile) is greater than that on the FLAC$^{3D}$, which in turn, leads to a higher $M_{\text{max}}$ in the model test.
Figure 6.12 Pile responses of Test F_4-3_50_60k_600
Figure 6.13 Force distribution from weight blocks to the pile head before and after the application of soil movement
6.5.4 Test on 32 mm pile at \( L_m/L_s = 200/500 \) and \( Q = 0 \) N

Two FLAC\(^3\)D analyses were performed to compare with the model test 2-5_32_0k_600_170304. The Mohr-Coulomb constitutive model, used in the analyses, had different dilation angles of \( \psi = 8^\circ \) and \( \psi = 38^\circ \), respectively. As shown in the parametric study (Section 6.4), the effect of the dilation angle is at a minimum for the analysis with the moving layer, \( L_m = 400 \) mm (in the standard model, \( L_m/L_s = 400/300 \)). However, with \( L_m = 200 \) mm (in the current model, \( L_m/L_s = 200/500 \)), as Figure 6.14 shows, the dilation angle has a significant effect on the bending moment, shear force, soil reaction, rotation and pile deflection profiles. These differences in the \( M_{\text{max}} \) between the model test and the FLAC\(^3\)D analyses are 232 % and 8 %, with the dilation angles of \( 8^\circ \) and \( 38^\circ \), respectively. Therefore, for \( L_m = 200 \) mm, the \( \psi = 38^\circ \) is in a good agreement with the model test. The differences are attributed to the apparent scale effects, which are often found in model tests. Turner and Kulhawy (1994) compared the drilled shaft side resistance in sand with both the model and full scale tests, and identified the effect of the dilation nature of sand and the curvature of the non-linear failure envelope (high friction angle) at a low confining stress, as two main causes of the apparent scale effects. This finding was evident in the present model tests as shown in Figures 6.15(a) and (b), for \( L_m \) of 200 mm and 400 mm, respectively. Thus, the degree of sand dilation depends on the thickness of \( L_m \) (or the overburden pressure at the sliding depth).
Figure 6.14 Pile responses of Test F_2-5_32_0k_600
Chapter 6: Three-dimensional finite difference analysis

Figure 6.15 Degree of sand dilating at different thickness of the moving layer ($L_m$)

(a) Sand dilated as $w_s$ increased at shallow $L_m$ ($L_m/L_s=200/500$)

(b) Sand dilation not evident at deeper $L_m$ ($L_m/L_s=400/300$)
6.5.5 Test on 50 mm pile at $L_m/L_s = 200/500$ and $Q = 0$ N

The effect of the dilation angle of sand was further investigated for the 50 mm diameter pile. Two analyses were performed on the models with $L_m/L_s = 200/500$, respectively. The dilation angles of sand used were: 1) $\psi = 8^\circ$; and 2) $\psi = 38^\circ$. Figure 6.16 presents the results from the FLAC$^{3D}$, together with the results from the model test. The differences in the $M_{\text{max}}$ between the FLAC$^{3D}$ and the model test are 387 % and 125 % for $\psi$ of $8^\circ$ and $38^\circ$, respectively. The deformation of the brick elements (or zones) at the “large stain” analysis (Itasca, 2002), enables the brick elements to deform, as the surrounding stress is applied on it. If the deformation was too large, and resulting the coordinates on the corners on one brick element coincided with each other, FLAC$^{3D}$ analysis would be terminated or stopped since the brick was in “illegal” geometry. This problem was only identified in the in large strain analysis, where the coordinates on the brick elements were free to move, and the coordinates were updated during the analyses. For the analysis with $\psi = 38^\circ$, the FLAC$^{3D}$ analysis was terminated at $w_s = 50$ mm. An illegal geometry was found from a brick element at the soil surface adjacent to the pile. The deformed brick element was small and had no significant impact on the pile response and so was removed; the analysis resumed up to $w_s = 120$ mm.

The magnitude of soil and pile displacements contour at the interval of 30 mm, 60 mm, 90 mm and 120 mm is shown in Figure 6.17. At $w_s = 30$ mm, the soil displacement at point A (immediately adjacent to the pile) is less than 30 mm. A further increase to $w_s = 120$ mm shows that the soil displacement, at point A, increases to approximately 50 mm (or $< w_s/2$). This finding is consistent with the investigation of the soil displacement in the model test (Subsection 3.2.2). The profile of the soil displacement, from the contour, is a wedge shape, which is also similar to a passive wedge of a retaining wall. Interestingly, the magnitude of the stress (x-direction) contour, shown in Figure 6.18, shows the magnitude of the stress as being a parabolic profile. At $w_s = 30$ mm, the high stress concentration is on the wall of the shear box, while at $w_s = 60$ mm, the stress starts to develop on the pile. The stress development can be identified as follows:

- at $w_s = 30$ mm, the stress builds up at the wall of the shear box
• at $w_s = 60$ mm, the stress starts to build up on the pile, where highest stress concentration is located in the $L_m$ layer

• at $w_s = 90$ mm, the stress continues to build up on the entire $L_m$ layer. It is also noticeable in $L_s$

• At $w_s = 120$ mm, the stress concentration contour builds up on both the $L_m$ and the $L_s$ layers, with the highest concentration at approximately the depth of $L_m/2$. In $L_s$ the stress distribution is similar to that of a rigid pile (Brinch Hansen, 1961; Meyerhof et al, 1981) with the point of rotation approximately 100 mm above the pile tip.
Figure 6.16 Pile responses of Test F_2-5_50_0k_600
Figure 6.17 Contour showing the magnitude of soil and pile displacement ($\psi = 8^\circ$)
Figure 6.18 The magnitude of stress (x-direction) build up in the shear box ($\psi = 8^\circ$)
6.6 The effect of $L_m/L_s$ on the pile deflection mode

A further study investigated the pile response with different ratios of $L_m/L_s$, not been previously used in the model tests reported in Chapters 4 and 5. It was quite straightforward to vary the $L_m/L_s$ in the FLAC$^{3D}$ model, when compared to physically varying it in the model test. Apart from the $L_m/L_s$, all the FLAC$^{3D}$ models used the same standard input parameters (shown previously in Tables 6.1). Six models using the 50 mm diameter pile and $Q = 0$ N were analysed on different ratios of $L_m/L_s$ (Table 6.5).

<table>
<thead>
<tr>
<th>Moving layer, $L_m$ (mm)</th>
<th>Stable layer, $L_s$ (mm)</th>
<th>Ratio, $L_m/L_s$</th>
<th>FLAC$^{3D}$ model reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>600</td>
<td>0.17</td>
<td>$F_{1-6}<em>{50}</em>{0k}$</td>
</tr>
<tr>
<td>200</td>
<td>500</td>
<td>0.40</td>
<td>$F_{2-5}<em>{50}</em>{0k}$</td>
</tr>
<tr>
<td>300</td>
<td>400</td>
<td>0.75</td>
<td>$F_{3-4}<em>{50}</em>{0k}$</td>
</tr>
<tr>
<td>400</td>
<td>300</td>
<td>1.33</td>
<td>$F_{4-3}<em>{50}</em>{0k}$</td>
</tr>
<tr>
<td>500</td>
<td>200</td>
<td>2.50</td>
<td>$F_{5-2}<em>{50}</em>{0k}$</td>
</tr>
<tr>
<td>600</td>
<td>100</td>
<td>6.00</td>
<td>$F_{6-1}<em>{50}</em>{0k}$</td>
</tr>
</tbody>
</table>

The bending moment profiles from all the FLAC$^{3D}$ models, as stated in Table 6.5, are presented in Figure 6.19(a). Generally, the shape of the bending moment profiles show a single curvature at $L_m \leq 200$ mm, and changes to a double curvature at $L_m/L_s$, in between 300/400 and 500/200. Finally, at $L_m/L_s = 600/100$, the bending moment profile shows a single curvature shape with the $M_{-\text{max}}$ of -40,374 Nmm at the depth of 400 mm. The magnitude of this negative moment is approximately the same as $M_{+\text{max}}$ obtained at $L_m/L_s = 300/400$. In summary, for a fixed pile length, by increasing the $L_m/L_s$ (from 0.17 to 6.00), the $M_{+\text{max}}$ and $M_{-\text{max}}$ change, but did not exceed $\pm$ 40,374 Nmm, respectively. However, the pile may not be stable when $L_m/L_s > 400/300$, due to the excessive pile deflection (> 50 mm or 1d) as shown on the deflection profiles of the pile in Figure 6.19(b).

The mode (translation, rotation or combination of both) of the deflection profiles changes as the $L_m/L_s$ increases. The maximum pile deflection, located at the surface, also increases with $L_m/L_s$ ratio. At $L_m/L_s = 500/200$, the soil movement is more than the pile deflection at the soil surface. Therefore at this ratio, the soil starts to flow past...
the pile at the soil surface. As the $L_m/L_s$ continues to increase, the soil layer, in which the soil movement exceeds the pile deflection, extends from the soil surface to a deeper depth within $L_m$. Starting from $L_m/L_s$ of about 500/200, the pile begins to “rip” through the soil at the stable layer ($L_s$), as the soil movement increases (the pile deflection mode consists of initially rotation, follows by translation as the $w_s$ increases).

Figure 6.19 Response of the 50 mm diameter pile with different $L_m/L_s$ ratio

(a) Bending moment profiles for different $L_m/L_s$ ratios

(b) Pile deflection profiles for different $L_m/L_s$ ratios
Chapter 6: Three-dimensional finite difference analysis

6.7 The effect of $L_m/L_s$ on the pile response

The effect of $L_m/L_s$ on the pile response was investigated by varying the $L_m/L_s$ of the standard model (see Table 6.1 for the input parameters) from 100/600 to 600/100. Three sets of analyses were performed and presented; an overview is presented in following:

1) the effect of $L_m/L_s$ on the pile response with $Q = 0$ N (Subsection 6.7.1)
2) the effect of $L_m/L_s$ on the pile response with $Q = 294$ N (Subsection 6.7.2)
3) the effect of the sand dilation angle on the pile response with different $L_m/L_s$, by increasing $\psi$ from $8^\circ(= \phi - 30^\circ)$ to $38^\circ(= \phi)$ (Subsection 6.7.3).

6.7.1 The effect of $L_m/L_s$ on the pile response with $Q = 0$ N

The maximum bending moments are obtained from the different ratios of $L_m/L_s$ at $w_s$ of 30 mm, 60 mm, 90 mm and 120 mm (Figure 6.20). Generally, at all $w_s$, the $M_{max}$ reaches the peak at the ratio $L_m/L_s$ of approximately unity. As shown in Figure 6.20(b), at $w_s = 60$ mm, a further increase of the $L_m/L_s$ ratio (>1) is achieved by increasing the $L_m$ and reducing $L_s$ (keeping the pile length constant), which causes the $M_{max}$ to be reduced from the peak, and subsequently there is a build up of the $M_{max}$, up to approximately -80 kNm at $L_m/L_s > 6.0$ (at $w_s = 120$ mm). At the $L_m/L_s$ of approximately 1.75, the $M_{max}$ becomes zero at $w_s = 30$ mm.

The maximum pile deflection, shear force and soil reaction against different ratios of $L_m/L_s$ are shown in Figures 6.21, 6.22 and 6.23, respectively. The significant differences are:

1) the maximum deflection as shown in Figure 6.21, shows differences between the FLAC$^{3D}$ and the model test of 84 % and 40 % at $w_s = 30$ mm and 120 mm, respectively. The “unbalanced force” (convergence criteria as discussed in Section 6.3) at $w_s \leq 30$ mm is relatively large when compared to that obtained for the higher $w_s$ (≥ 60 mm). Thus the model is not fully converged at low the $w_s$.

2) the maximum shear force, at approximately $L_m/L_s = 1.0$, as shown in Figures 6.22(a) to (c), show a large difference between the FLAC$^{3D}$ and the experimental results at $w_s = 30$ mm~ 90mm. This difference is resulted from
the effect of the dilatancy of the sand, as previously described in Subsections 6.5.4 and 6.5.5.

3) the maximum soil reactions, at approximately $L_m/L_s = 1.0$, as shown in Figures 6.23(a) to (d), show a large difference between the FLAC$^{3D}$ and the experimental results at $w_s = 30\, \text{mm} \sim 90\, \text{mm}$. As noted previously, in relation to the maximum shear force, this difference is again resulted from the effect of dilatancy of the sand.

The FLAC$^{3D}$ results for the maximum bending moment, pile deflection, shear force and soil reaction against $L_m/L_s$, are shown in Figures 6.20(e), 6.21(e), 6.22(e) and 6.23(e), respectively. For each $w_s$, there is a unique profile. The peak value increases as the $w_s$ increases. As expected, this response is quite different from that noted in the experimental results. For example, on the model tests with $L_m/L_s = 200/500$ (see Chapter 4, Figure 4.9(b)), a definite peak value of $M_{r\text{max}}$ is reached at $w_s = 20\, \text{mm}$. The difference between the FLAC$^{3D}$ and the model test can be attributed to modelling the sand as continuum elements in FLAC$^{3D}$, where in reality, the sand is made of particles, which start to flow around the pile, when the applied $w_s$ is large enough. This limitation and possible improvement will be discussed later.
Figure 6.20 Comparison of $M_{\text{max}}$ at $Q = 0$ N between FLAC$^{3\text{D}}$ and Experimental results: (a) $w_s = 30$ mm; (b) $w_s = 60$ mm; (c) $w_s = 90$ mm; (d) $w_s = 120$ mm; and (e) Summary of FLAC$^{3\text{D}}$ analysis
Figure 6.21 Comparison of maximum pile deflection at \( Q = 0 \) N between FLAC\textsuperscript{3D} and Experimental results: (a) \( w_s = 30 \) mm; (b) \( w_s = 60 \) mm; (c) \( w_s = 90 \) mm; (d) \( w_s = 120 \) mm; and (e) Summary of FLAC\textsuperscript{3D} analysis
Figure 6.22 Comparison of maximum shear force at $Q = 0$ N between FLAC$^{3D}$ and Experimental results: (a) $w_s = 30$ mm; (b) $w_s = 60$ mm; (c) $w_s = 90$ mm; (d) $w_s = 120$ mm; and (e) Summary of FLAC$^{3D}$ analysis.
Figure 6.23 Comparison of maximum soil reaction at Q = 0 N between FLAC³D and Experimental results: (a) ws = 30 mm; (b) ws = 60 mm; (c) ws = 90 mm; (d) ws = 120 mm; and (e) Summary of FLAC³D analysis.
6.7.2 The effect of \( L_m/L_s \) on the pile response with \( Q = 294 \text{ N} \)

The \( M_{\text{max}} \) reaches the peak at approximately \( L_m/L_s \) of 1.0, as shown in Figure 6.24. Beyond \( L_m/L_s > 1.0 \), \( M_{\text{max}} \) starts to reduce, while the \( M_{\text{max}} \) starts to increase, and becomes negative as the \( L_m/L_s \) increases further (> 2.7). A comparison of the FLAC\(^{3D} \) results with the experimental results, at \( L_m/L_s = 200/500 \), showed that the difference in the \( M_{\text{max}} \) is 387 \% (Figure 6.24(b)). This difference once again highlights the effect of the dilation angle of sand on the pile response, especially at a low \( L_m \), as previously discussed in Subsections 6.5.4 and 6.5.5. In general, the \( M_{\text{max}} \) from the analyses with \( Q = 294 \text{ N} \) are higher when compared with the analyses with \( Q = 0 \text{ N} \) (see Figure 6.20).

Generally, the \( M_{\text{max}} \) and \( M_{\text{max}} \) from the FLAC\(^{3D} \) show a good agreement with that from the experimental results at both the \( L_m/L_s \) of 200/500 and 400/300, respectively. At \( w_s = 30 \text{ mm} \), the difference between the FLAC\(^{3D} \) and the experimental results is approximately 4.5 times. One possible cause for the large difference could be the effect of the “unbalanced force” at \( w_s = 30 \text{ mm} \), which is relatively large. This “unbalanced force” is only reduced at \( w_s > 40 \text{ mm} \), however, at \( w_s = 60 \text{ mm} \sim 120 \text{ mm} \), a smaller difference is noted in the \( M_{\text{max}} \) between the FLAC\(^{3D} \) and the experimental results.

The recorded maximum pile deflections from the FLAC\(^{3D} \) and the experimental results show a good agreement (Figure 6.25). The maximum pile deflections generally show a maximum difference (between FLAC\(^{3D} \) and the experimental results, Figure 6.25(a)) of 109 \% and 9 \% for \( L_m/L_s \) of 200/500 and 400/300, respectively. The pile deflection increases with the increase of \( L_m/L_s \). However, this rate of increments varies and peaks in between the \( L_m/L_s \) of 200/500 and 400/300. Furthermore, the maximum pile deflections are higher, when compared to those piles without an axial load (Figure 6.21).

The maximum shear force and soil reaction against the different ratios of \( L_m/L_s \) are shown in Figures 6.26 and 6.27, respectively. The differences between the FLAC\(^{3D} \) and the experimental results are:
• the maximum pile deflection increases significantly between $\frac{L_m}{L_s}$ of 0 to 2.0, and, at $\frac{L_m}{L_s} > 2.0$, the rate of this increment is reduced. Subsequently, between the $\frac{L_m}{L_s}$ of 2.0 and 6.0, for $w_s = 30$ mm ~ 120 mm, the differences in the maximum pile deflection did not exceed 10%.

• the maximum shear force and the maximum soil reaction show a good agreement at $\frac{L_m}{L_s} = 400/300$, but not for $\frac{L_m}{L_s} = 200/500$. 
Figure 6.24  Comparison of $M_{\text{max}}$ at $Q = 294$ N between FLAC$^{3D}$ and Experimental results: (a) $w_s = 30$ mm; (b) $w_s = 60$ mm; (c) $w_s = 90$ mm; (d) $w_s = 120$ mm; and (e) Summary of FLAC$^{3D}$ analysis
Figure 6.25 Comparison of maximum pile deflection at $Q = 294$ N between FLAC\textsuperscript{3D} and Experimental results: (a) $w_s = 30$ mm; (b) $w_s = 60$ mm; (c) $w_s = 90$ mm; (d) $w_s = 120$ mm; and (e) Summary of FLAC\textsuperscript{3D} analysis.
Figure 6.26 Comparison of maximum shear force at $Q = 294 \text{ N}$ between FLAC$^{3D}$ and Experimental results: (a) $w_s = 30 \text{ mm}$; (b) $w_s = 60 \text{ mm}$; (c) $w_s = 90 \text{ mm}$; (d) $w_s = 120 \text{ mm}$; and (e) Summary of FLAC$^{3D}$ analysis
Figure 6.27 Comparison of maximum soil reaction at \( Q = 294 \) N between FLAC\(^{3D}\) and Experimental results: (a) \( w_s = 30 \) mm; (b) \( w_s = 60 \) mm; (c) \( w_s = 90 \) mm; (d) \( w_s = 120 \) mm; and (e) Summary of FLAC\(^{3D}\) analysis
6.7.3 The effect of sand dilation angle on the pile response different L_m/L_s ratios

The two cases previously discussed in Subsections 6.5.4 and 6.5.5, with L_m/L_s=200/500, have shown the dilation angle of sand has a significant effect on the bending moment profile of the pile. However, as shown in Subsection 6.5.3, this effect on the model at L_m/L_s = 400/300 is not as significant. In order to further investigate the effect of the sand dilation angle on the FLAC3D model, additional analyses (Table 6.5) were performed using a sand dilation angle of $\psi = 38^\circ$ (also know as the associated flow rule). Figure 6.28 shows the comparison, in terms of M_{max}, for the different L_m/L_s ratios between the two sand dilation angles of $\psi = 8^\circ$ and $38^\circ$.

Briefly, the comparison between the FLAC3D analyses with $\psi = 8^\circ$ and $38^\circ$ shows that:

- in general, the M_{+max} reaches its peak at L_m/L_s, approximately 1.0, with the maximum difference being 63% (at $w_s = 120$ mm)
- $\psi$ has an effect on the M_{+max} at L_m/L_s < 2.0, while beyond L_m/L_s >2.0 the difference in the M_{+max} become insignificant
- the difference in M_{-max} increases with L_m/L_s ratio.

The comparison between FLAC3D analyses and the model tests indicates that:

- in general, FLAC3D results, with $\psi = 38^\circ$, show a better prediction for both L_m/L_s = 200/500 and 400/300
- the M_{+max} from FLAC3D is lower than that observed from the model tests (owing to the difference in the p-delta effect being considered, see Figure 6.13)
- the FLAC3D results show a better agreement with that of the model tests at $w_s \geq 60$ mm
- at $w_s = 60$ mm and 120 mm, the M_{max} from the model tests are in between the FLAC3D results of $\psi = 8^\circ$ and $38^\circ$, respectively.

From these findings, the effect of the sand dilation angle on the M_{+max} and the M_{max} is shown to depend on the L_m/L_s. For example, as shown in Figure 6.28(b), with L_m/L_s =
5.0, the majority of the pile length is in the $L_m$ layer, while the dilation angle has a significant effect on the $M_{\text{max}}$. On the other hand, with $L_m/L_s = 1.0$, the majority of the pile length is in the $L_s$ layer, while the dilation angle has a significant effect on the $M_{\text{max}}$.

It is envisaged that, in the $L_s$ layer, most of the soil elements are in the elastic state, in particular at a low $w_s$ ($< 30 \text{ mm}$) and $L_m/L_s$ ratio ($< 1.0$). Therefore, the behaviour of the pile is largely governed by the elastic parameters of the soil, such as the Poisson’s ratio and the Young’s modulus. As previously shown in Subsection 6.4.3.1, the selection of $E_s$ has an effect on the $M_{\text{max}}$, while the current model using a constant $E_s$ for the entire depth of the model, possible improvements could be made to the current model with: $E_s$ varies with the depth; and $E_s$ changes with the soil strain or/and stress.
Figure 6.28 The effect of sand dilation on the $M_{\text{max}}$ at different $L_m/L_s$ ratios: (a) $w_s = 20$ mm; (b) $w_s = 60$ mm; (c) $w_s = 90$ mm; and (d) $w_s = 120$ mm; and (e) summary of FLAC$^{3D}$ with $\varphi = 38^\circ$
Chapter 6: Three-dimensional finite difference analysis

6.8 On $M_{\text{max}}$ against $S_{\text{max}}$ relationship obtained from the numerical analysis

With increasing $L_{\text{m}}/L_s$

Figure 6.29 shows the $M_{\text{max}}$ against $S_{\text{max}}$ for the Flac$^{3D}$ models having $L_{\text{m}}/L_s$ of 100/600 to 600/100 (see Table 6.5). In general, as listed in Table 6.6, the gradient of the lines ($\alpha$) 1 to 4 reduces with the increase of $L_{\text{m}}/L_s$, until $L_{\text{m}}/L_s \leq 400/300$. It should be noted that $\alpha$ of 0.14 (line 5) and 0.25 (line 6), for the models with $L_{\text{m}}/L_s$ of 500/200 and 600/100 respectively, is correct, considering the negative magnitude of both the $M_{\text{max}}$ and the $S_{\text{max}}$ (see Figure 6.19(a) for the bending moment profile). The lines 1 to 4 are seen to “rotate” around the origin in an anticlockwise direction, as $L_{\text{m}}/L_s$ increases. This relationship is associated with the progressive change of the pile deformation mode as $L_{\text{m}}/L_s$ increases, as previously discussed in Section 6.6.

![Figure 6.29](image)

In general, the relation $M_{\text{max}} = (0.14 \sim 0.25)S_{\text{max}}$, fits well the FLAC$^{3D}$ results with the different $L_{\text{m}}/L_s$ (Table 6.6).

<table>
<thead>
<tr>
<th>$L_{\text{m}}/L_s$</th>
<th>100/600</th>
<th>200/500</th>
<th>300/400</th>
<th>400/300</th>
<th>500/200*</th>
<th>600/100*</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha$</td>
<td>0.17</td>
<td>0.19</td>
<td>0.16</td>
<td>0.14</td>
<td>0.14</td>
<td>0.25</td>
</tr>
<tr>
<td>Line</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
</tr>
</tbody>
</table>

*Note: *Both $S_{\text{max}}$ and $M_{\text{max}}$ are negative*
Chapter 6: Three-dimensional finite difference analysis

With two different sand dilation angles

The $M_{\text{max}}$ against $S_{\text{max}}$ for the two sets of the Flac\textsuperscript{3D} models, with $\psi$ of $8^\circ$ and $38^\circ$ are shown in Figure 6.30. The $M_{\text{max}}$ and $S_{\text{max}}$ are taken at $w_s = 60$ mm. The $M_{\text{max}}$ against $S_{\text{max}}$ relation is found only for the models with $L_m/L_s$ of $100/600$ to $400/300$, as was previously shown in Figure 6.29, with the positive $M_{\text{max}}$ and $S_{\text{max}}$ (at $L_m/L_s \leq 400/300$).

![Graph showing $M_{\text{max}}$ against $S_{\text{max}}$ for Flac\textsuperscript{3D} models.]

Figure 6.30 $M_{\text{max}}$ against $S_{\text{max}}$ for two sets of Flac\textsuperscript{3D} models: 1) $\psi = \phi$; and 2) $\psi = \phi - 30^\circ$ ($w_s = 60$ mm, $L_m/L_s = 100/600$~$300/200$)

The $M_{\text{max}}$ against $S_{\text{max}}$ relationship, previously discussed in Chapter 4 (experimental results), showed that $M_{\text{max}} = \alpha S_{\text{max}}$. The $\alpha$, obtained herein for the Flac\textsuperscript{3D} models, with $\psi = \phi$ and $\psi = \phi - 30^\circ$, are 0.20 and 0.14, respectively. It can be seen that the $\alpha$ increases with the dilation angle of sand, and the range of $\alpha$ (0.14 to 0.20) found is consistent with that proposed by Guo and Qin (2009) of 0.103 to 0.28.
6.9 Comparison between prediction of FLAC3D and an existing solution

The following section presents an overview of a comparison between the prediction of FLAC3D and an existing solution proposed by Chen and Poulos (1997).

6.9.1 Chen and Poulos (1997)

Chen and Poulos (1997) presented a series of design curves for the analysis of single piles and pile groups subjected to either the rectangular or the triangular soil movement profiles. These design curves, showing the relationship between normalised $M_{\text{max}}$ and $L_w/L$, are obtained from the elastic analysis (on the soil and the pile) with the boundary element method. A series of case studies (including the model tests by Chen (1994)) shows that these design curves tended to give an upper bound solution for the maximum bending moment on the pile.

In the current study, the input parameters, used to select a design curve for the comparison, are: $E_p = 70,000$ MPa; $I_p = 84,113$ mm$^4$; $L = 700$ mm; and $N_h = E_s/L = 8.2 \times 10^{-4}$ MPa/mm. These parameters give approximately: $K_R = E_p I_p / E_s L \approx 10^{-2}$; and $L/d \approx 10$. In addition, the following factors in Chen and Poulos (1997) may have influenced the comparison:

- the boundary element method analysis assumed that the soil is an elastic material (although the yield strength of soil can be specified), while in the FLAC3D analyses it is taken as an elastic-plastic material using the Mohr-Coulomb constitutive model
- the design curve is selected from the nearest values of $K_R$ and $L/d$.
- the design curve are plotted on a logarithmic scale. For comparative purposes, the results from the FLAC3D, in terms of $M_{\text{max}}$ and $M_{\text{-max}}$, are plotted on this curve as absolute values, giving $M_{\text{+max}} = |M_{\text{+max}}|$ and $M_{\text{-max}} = |M_{\text{-max}}|$
- Figure 6.31 presents the normalised FLAC3D results (Figure 6.20(e), at $w_s = 120$ mm) and the design curve. The $|M_{\text{+max}}|$ reaches the peak at approximately $L_m/L$ of 0.35 and 0.45, for the design curve and FLAC3D, respectively
the design curve gives a higher value of $|M_{\text{max}}|$ up to $L_m/L_s \leq 0.7$. At above $L_m/L_s > 0.7$, the design curve seems to have under predicted the results from the FLAC$^{3D}$ (where, $|M_{\text{max}}| > |M_{\text{max}}|$)

in general, the results from the design curve and FLAC$^{3D}$ are in good agreement.

Figure 6.31 Comparison between FLAC$^{3D}$ and Chen & Poulos (1997)
6.10 Limitation and possible improvement on the FLAC$^{3D}$ model

Throughout the current numerical study, a number of limitations and possible improvements to the current Flac$^{3D}$ models have been identified. Where possible, and within the time frame, attempts to address these limitations and implement improvements to the models have been carried out. Nevertheless, a number of limitations and improvements are yet to be addressed. These are identified below as a guide to important future research directions:

- one limitation, in the current and other three dimensional modellings, was the computing capacity, which resulted in time consuming analyses
- in using the large-strain mode in FLAC$^{3D}$, the brick elements (or zones) were allowed to deform with the applied soil movement. Regretably, the allowed maximum strain or deformation was limited. Indeed, the continuum model was only representative at the lower $w_s$, while, at the higher $w_s$, when the sand starts to “flow” around the pile, the problem becomes more of a particles flow problem. Hence, the discontinuum program such as PFC$^{3D}$ may be more appropriate for the modelling of the flow of sand particles and its interaction with pile
- as shown by Bolton (1986) and Taylor (1948), the confining stress and relative density of the sand had effect on both the friction and the dilation angles of the sand. In the current study, the different confining pressures were anticipated (at the interface between $L_m$ and $L_s$) as a result of the models of the different $L_m/L_s$ ratios. The friction and the dilation angles required variations, according to the confining pressure (of thickness of $L_m$). Furthermore, in the Mohr-Coulomb constitutive model, both $\phi$ and $\psi$ were independent of the confining pressure and the relative density of the sand. Therefore, the prediction of the FLAC$^{3D}$ may be improved by incorporating the variation of $\phi$ and $\psi$ manually, according to the $L_m/L_s$ ratio of the model. Chen et al (2003) demonstrated that in the FLAC$^{3D}$, the correct selection of $\phi$ and $\psi$, with respect to the confining pressure, lead to a good agreement between the experimental and the numerical model
- as only two constitutive models were used, perhaps, it would be worth investigating other constitutive modes, such as those by Ladd and Duncan
Chapter 6: *Three-dimensional finite difference analysis*

(1975), and Ladd (1977), which took into account the effects of the confining pressure on the strength of sand (or \( \phi \))

- the current model did not investigate the effect of using the interface elements between \( L_m \) and \( L_s \) layers, which may have been more representative of the model test. While several attempts were made to include this interface, they were not successfully incorporated into the model.

6.11 Conclusions

FLAC\(^{3D}\) program was successfully used to model the results from the single pile model tests that are associated with large deformations due to the moving soil. With the large strain mode, the soil was modelled as elements or zones, which were allowed to deform with the change of the surrounding stresses. The results from the analyses show the following key findings:

- Young’s modulus, friction angle and dilation angle of the sand had a significant effect on the behaviour of the pile, in particular the \( M_{\text{max}} \). On the other hand, the interface properties, density of the brick elements and constitutive model had a less significant effect on the pile behaviour
- the shape of the bending moment, shear force, soil reaction, pile rotation and deflection profiles from the FLAC\(^{3D}\) were in good agreement to that obtained from the experimental results. In all the analyses, the \( M_{\text{max}} \) (with \( \psi = \phi \) and \( \psi = \phi - 30^\circ \)) obtained from the FLAC\(^{3D}\) were lower than those from the experimental results
- the comparison between the FLAC\(^{3D}\) and the experimental results revealed the effect of the initial pile verticality, and the way the axial load was applied on the pile head, caused different degree of p-delta effect on the pile, especially for the test with \( Q = 589 \) N
- with a low sliding depth (\( L_m \leq 200 \) mm), and thus a low confining pressure, using \( \psi = \phi \), predicted closer outcome for the experimental results than when using \( \psi = \phi - 30^\circ \)
- the mode of the deflection profiles changed as the \( L_m/L_s \) increased. The maximum pile deflection, located at the surface, increased with the \( L_m/L_s \)
Beyond the $L_m/L_s > 500/200$, the maximum pile deflection at the surface exceeded the applied $w_s$

- the $M_{\max}$ peaked at $L_m/L_s$ close to unity
- the FLAC$^{3D}$ results showed the $M_{\max}$ increased with $w_s$, while the experimental results indicated that, at $w_s > 20$ mm ($L_m/L_s = 200/500$), the $M_{\max}$ remains constant, reflecting the impact of sand particles flow in reality
- the sand dilation angle had the greatest effect on the $M_{+\max}$ at $L_m/L_s < 1.0$, with a negligible effect at $L_m/L_s > 2.0$
- the relation $M_{\max} = (0.14 ~ 0.25)S_{\max}$, fitted well the FLAC$^{3D}$ results with different $L_m/L_s$, which was consistent with that proposed by Guo and Qin (2009).
Chapter 7: Single Pile and Pile Group Tests in Clay

7.0 Introduction

A number of studies on laboratory model tests have investigated the behaviour of piles subjected to lateral soil movements in clay. These tests can be summarised into two categories, namely:

1) piles subjected to lateral soil movement caused by landslides, tunneling operations and deep excavation (Pan, 1998; Loganathan; 1999; Leung et al, 2001)
2) piles subjected to lateral soil movement caused by adjacent surcharge or embankment weight (Springman, 1989; Steward, 1992; Bransby, 1995; Ellis, 1997).

In this chapter, an overview of the typical test results are presented for single piles and pile groups embedded in clay subjected to rectangular lateral soil movement. No axial load was applied onto the piles in the tests. Indeed, the tests conducted focused on investigating the effect of the pile diameter, the ratio of L_m/L_s, the pile group arrangement and the pile group spacing.

7.1 Test description

Both single piles and pile group tests used laboratory prepared kaolin clay. The clay sample preparation and the testing procedure were slightly different from those described in Chapter 3 (piles in sand). The following subsections highlight the modifications made to the shear box and the experimental procedure.

7.1.1 Shear box

The shear box had internal dimensions of 0.5 m by 0.5 m, and a height of 0.8 m (see Figure 7.1). The shape of the shear box was identical to the shear box used in the sand testing, except for its smaller plan dimensions. The boundary distance (measured from the pile face to the shear box inner wall) to the pile diameter ratio was kept to approximately 4.5, which was higher than the minimum value of 4 proposed by
Bolton and Gui (1993) in order to prevent any significant boundary effect onto the piles.

The upper movable part of the box consisted of a number of 25 mm thick square laminar aluminium frames, stacked together, to achieve a thickness of $L_m \leq 400$ mm. They were moved together by a rectangular loading block in order to generate a uniform lateral soil movement. The rate of this movement was controlled by a hydraulic pump, and a flow control valve. To achieve a stable clay layer of thickness $L_s \geq 400$ mm, the lower fixed section consisted of a timber box 400 mm in height, and a desired number of laminar aluminium frames.
Figure 7.1 Dimensions of shear box used for pile testing in clay
7.1.2 Clay properties and preparation procedure

The clay used in the model tests was the commercially available kaolin from the Skardon River, Cape York Peninsula, Australia. The kaolin, called Microbrite\textsuperscript{®} CH80, and its properties are listed in Table 7.1.

Table 7.1 Properties of kaolin clay

<table>
<thead>
<tr>
<th>Properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid limit (%)</td>
<td>42.08</td>
</tr>
<tr>
<td>Plastic limit (%)</td>
<td>25.54</td>
</tr>
<tr>
<td>Specific gravity, $G_s$</td>
<td>2.64</td>
</tr>
<tr>
<td>Average moisture content (%)</td>
<td>39.2</td>
</tr>
<tr>
<td>Average density (kN/m\textsuperscript{3})</td>
<td>17.5</td>
</tr>
</tbody>
</table>

The clay, supplied in a spray dried powder form, was uniformly mixed with a predetermined amount of water in a concrete mixer to achieve a moisture content close to its liquid limit. As a large amount of clay needed for the experiments, immediately after mixing, the clay was stored in storage boxes and wrapped with plastic cling wrap to prevent moisture lost. The duration between the first mix and the last mix was approximately 4 days. The clay was left in the storage boxes for a period of 14 days before being placed into the shear box. Air voids were avoided by the pouring technique, and by choosing a moisture content close to the liquid limit. Once the shear box was filled, the surface of the clay, inside the shear box and the outside perimeter of the shear box, was covered with two to three layers of plastic cling wrap to prevent moisture lost. The clay was again left for 24 hours inside the shear box before the pile was driven (by hand) into position and tested. In between the tests (a minimum 24-hours apart), the clay was left in the shear box, and covered with plastic cling wrap. It should be noted that no preconsolidation pressure was applied to the clay before or after each test. The model tests using unconsolidated clay were also performed on the laterally loaded piles as described by Ranjan et al (1977), Allen and Reese (1980), and Smith (1983).
7.1.3 Model pile

The model piles were identical to those used for the tests in sand (d = 25 mm and 32 mm, see Chapter 3). However, in clay testing an extra layer of plastic electrical tape was wrapped around the pile to prevent any moisture, from the clay, getting into the strain gauge connections. It should be noted that, both piles in the pile group tests were instrumented with strain gauges, and identified as piles ‘A’ and ‘B’, depending on the location of the piles. The pile caps used in the clay tests (1 × 2 and 2 × 1 pile groups) were same as those used earlier, and described in Chapter 5.
7.1.4 Details of the tests performed

A numbering system similar to that used in Chapters 4 and 5 was used to identify the tests in clay. Figure 7.2 shows the test identification number tests in clay. At the front of the number, an additional alphabet ‘C’ is included to identify those tests conducted in clay.

![Diagram showing single pile and pile group tests in clay](image)

(a) Information about the pile tests

(b) Single pile test

(c) Pile group test

Figure 7.2 Test identification number for single piles and pile groups in clay

A total of six tests were conducted in clay; the details of the parameters selected for each test are summarized in Table 7.2. The single piles were tested without any constraint, except for soil resistance. On the other hand, the piles in the pile group tests were connected using a pile cap, at the pile head, located 500 mm above the clay surface. A uniform soil movement was applied using the rectangular loading block at an increment of 10 mm until a total of 110 ~ 130 mm was reached. The average rate of the soil movement (taking into account the time taken for the data recording for each 10 mm increment of $w_s$) was approximately 2 mm per minute. Strain gauge
readings were used to obtain the bending moment profile along the pile for each increment of soil movement. Other pile responses, in terms of shear force, soil reaction, pile rotation, and pile deflection were derived from the bending moment; they are presented and discussed below.

Table 7.2 Tests on single piles and pile groups in clay subjected to uniform soil movement profile

<table>
<thead>
<tr>
<th>Pile arrangement</th>
<th>L_m (mm)</th>
<th>L_s (mm)</th>
<th>d (mm)</th>
<th>Pile spacing</th>
<th>Pile</th>
<th>Test identification number</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single piles</td>
<td>200</td>
<td>500</td>
<td>25</td>
<td>-</td>
<td>-</td>
<td>C_2-5_25_170805</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>300</td>
<td>32</td>
<td>-</td>
<td>-</td>
<td>C_2-5_32_250605</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>C_4-3_32_290705</td>
</tr>
<tr>
<td>Piles in a row (1×2)</td>
<td>200</td>
<td>500</td>
<td>32</td>
<td>3d</td>
<td>A</td>
<td>C_2-5_32_1×2_3d_A_230705</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>500</td>
<td>32</td>
<td>5d</td>
<td>B</td>
<td>C_2-5_32_1×2_5d_B_230705</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>A</td>
<td>C_2-5_32_1×2_5d_A_300705</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>B</td>
<td>C_2-5_32_1×2_5d_B_300705</td>
</tr>
<tr>
<td>Pile in a column (2×1)</td>
<td>200</td>
<td>500</td>
<td>32</td>
<td>3d</td>
<td>A</td>
<td>C_2-5_32_2×1_3d_A_270705</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>B</td>
<td>C_2-5_32_2×1_3d_B_270705</td>
</tr>
</tbody>
</table>
7.2 Determination of undrained shear strength

The undrained shear strength of clay was assessed by the vane shear tests after the completion of each test. Figure 7.3 shows the locations, in the shear box, in which the vane shear tests were performed. The average undrained shear strengths, at the respective depths inside the shear box, obtained from the tests, are shown in Figure 7.4.

Figure 7.3 Locations of vane shear tests in the shear box

Figure 7.4 Average shear strength of clay in the shear box
7.3 Single pile tests

Three single pile tests were undertaken having different pile diameters and the \( L_m/L_s \) ratios. The test results are presented in the following subsections.

7.3.1 Test C_2-5_25_170805

Figure 7.5 shows the response of the 25 mm diameter pile with \( L_m/L_s \) of 200/500. At \( w_s = 130 \) mm, the maximum bending moment \( (M_{\text{max}}) \) of 4,484 Nmm is located at a depth of 360 mm (> \( L_m \)). At this depth the shear force is indeed zero. The maximum shear force and the maximum soil reaction are 20.7 N and 0.089 N/mm, respectively. The bending moment, shear force, soil reaction, pile rotation and pile deflection increase with \( w_s \) at a constant rate. The pile deflection profiles show the pile rotated at the pile tip at \( w_s \) of 10 mm to 30 mm. Further increase of \( w_s \) (> 30 mm) shows that the point of rotation increases to a depth of 500 mm. The rotation profiles shown in Figure 7.5(d) indicate a rigid pile.

7.3.2 Test C_2-5_32_250605

Figure 7.6 presents the five responses of the 32 mm diameter pile with \( L_m/L_s \) of 200/500. The bending moment, shear force, soil reaction, pile rotation and pile deflection increase constantly with \( w_s \). As can be seen in Figure 7.6(a), at \( w_s = 110 \) mm, the maximum bending moment of 5,882 Nmm is at the depth of 360 mm (within \( L_s \)). At this depth the shear force is indeed zero (Figure 7.6(b)). Figure 7.6(e) indicates that the pile rotates initially, at a depth of approximately 360 mm \( (w_s = 10 \text{ to } 30 \text{ mm}) \). With a further increase in the soil movement \( (w_s > 30 \text{ mm}) \), the pile rotates at a lower depth of 520 mm. At \( w_s = 110 \text{ mm} \), the pile deflection at the soil surface is 69.7 mm, which indicates that the soil has flowed around the pile \( (w_s > \text{pile deflection}) \). The rotation profile (Figure 7.6(d)) shows that the rotation is constant with depth, therefore, the pile is seen as a rigid pile.
7.3.3 Test C_4-3_32_290705

Figure 7.7 presents the pile response of the 32 mm with $L_m/L_s$ of 400/300. The bending moment and the pile deflection profiles, shown in Figures 7.7(a) and (e) respectively, indicate that the pile is bending toward the direction of the soil movement. The $M_{\text{max}}$ recorded are consistently at a depth of 60 mm for the entire range of the soil movement ($w_s = 10 \sim 130$ mm). Generally, the bending moments, recorded at the surface, are approximately 22.4 % of the $M_{\text{max}}$. The deflection profiles indicate that the pile rotates at a fixed point located at about a depth of 500 mm. The maximum pile deflection of 120 mm is very close to the corresponding $w_s$ of 130 mm. The constant rotation with depth indicated that the pile is a rigid pile.

A comparison made with the previous Test C_2-5_32_250605 (with $L_m/L_s = 200/500$, Figure 7.6) shows the following effect of the $L_m/L_s$ ratio on the pile responses:

- the shape of the bending moment profiles are different between the tests
- the maximum pile deflection (at surface) increases with $L_m/L_s$ ratio.
Figure 7.5 Pile response of Test C_2-5_25_170805
Figure 7.6 Pile response of Test C_2-5_32_250605
Figure 7.7 Pile response of Test C_4-3_32_290705
7.3.4 Summary of the single pile tests

The critical pile responses of the single pile tests are summarised in Table 7.3. It can be seen that the depth of $M_{\text{max}}$ reduces as $L_m/L_s$ increases. It should be noted that the $c_u$ in each test are different, therefore, a direct comparison of the pile responses is not possible without taking into account of the different $c_u$ on each test. In order for the comparison to be made, the $M_{\text{max}}$ is normalized with $c_u d^3$. Figure 7.8 shows $M_{\text{max}}/c_u d^3$ plotted against the pile diameter. On the tests with $L_m/L_s = 200/500$, at $w_s$ of 60 mm and 120 mm, $M_{\text{max}}/c_u d^3$ increases with the pile diameter.

<table>
<thead>
<tr>
<th>Test number</th>
<th>Depth of $M_{\text{max}}$ (mm)</th>
<th>$M_{\text{max}}/c_u d^3$</th>
<th>Max. shear force, $S_{\text{max}}$ (N)</th>
<th>Max. pile deflection (mm)</th>
<th>Max. soil reaction, $r_{\text{max}}$ (N/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C_2-5_25_170805</td>
<td>360</td>
<td>9.8</td>
<td>-10.1</td>
<td>31.8</td>
<td>0.05</td>
</tr>
<tr>
<td>C_2-5_32_250605</td>
<td>360</td>
<td>20.4</td>
<td>-16.2</td>
<td>69.7</td>
<td>0.03</td>
</tr>
<tr>
<td>C_4-3_32_290705</td>
<td>60</td>
<td>2.3</td>
<td>5.4</td>
<td>56.2</td>
<td>-0.02</td>
</tr>
</tbody>
</table>

Figure 7.8 $M_{\text{max}}/c_u d^3$ against pile diameter ($L_m/L_s = 200/500$)
7.4 Pile group tests

After the completion of the single pile tests, two pile group tests were carried out. The number of piles used in the pile group tests was limited to two piles in a group, because of the size limitation of the shear box. Further, to prevent any potential boundary effect, a clear distance between the pile and the shear box wall was set at not less than 4d or 138 mm (Figure 7.1(c)). All the pile group tests used 32 mm diameter piles at \( L_m/L_s = 200/500 \). The pile groups were subjected to a maximum soil movement of 130 mm, applied at 10 mm intervals. A pile cap was used to connect the piles at the pile heads located 500 mm above the soil surface.

7.4.1 Test C_2-5_32_1×2_3d_230705

Pile group with the piles arranged in a row (1 \( \times \) 2) was tested. Figures 7.9 and 7.10 show the responses of piles A and B of the 1 \( \times \) 2 pile group test, respectively. Their maximum bending moment, maximum shear force and maximum soil reaction increase with the soil movement. The pile response at \( w_s = 130 \) mm is summarised in Table 7.4.

<table>
<thead>
<tr>
<th>Pile response</th>
<th>Pile A</th>
<th>Pile B</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{+\text{max}} ) (Nmm)</td>
<td>5,914</td>
<td>8,240</td>
<td>39%</td>
</tr>
<tr>
<td>( S_{-\text{max}} ) (N)</td>
<td>-23.02</td>
<td>-32.13</td>
<td>40%</td>
</tr>
<tr>
<td>( r_{+\text{max}} ) (N/mm)</td>
<td>0.10</td>
<td>0.13</td>
<td>30%</td>
</tr>
<tr>
<td>Deflection at surface (mm)</td>
<td>81.8</td>
<td>81.8</td>
<td>0%</td>
</tr>
</tbody>
</table>

The difference in the \( M_{+\text{max}}, S_{-\text{max}} \) and \( r_{+\text{max}} \) between piles A and B is in the range of 30% to 40%. As the pile cap connects both piles, the same deflections are recorded at the soil surface. Figures 7.9(e) and 7.10(e) present that the deflection profiles, where the piles consist of both a translation and rotation. Furthermore, the pile rotation profiles (Figures 7.9(d) and 7.10(d)) show a constant rotation angle with depth, indicating that piles A and B behave as rigid piles.
Figure 7.9 Pile response of Test C_2-5_32_1×2_3d_A_230705 (pile A)
Figure 7.10 Pile response of Test C_2-5_32_1×2_3d_B_230705 (pile B)
7.4.2 Test C_2-5_32_1×2_5d_300705

The pile group test was performed using the same piles in a row arrangement, as previously described, but with the pile spacing increased to five times the pile diameter (5d). Figures 7.11 and 7.12 show the response of piles A and B, respectively. It can be seen that the depth of the $M_{\text{max}}$ of piles A and B is 360 mm (within $L_s$). Table 7.5 provides a comparison between piles A and B at $w_s = 130$ mm, where the difference in the $M_{\text{max}}$ between piles A and B is 45%. This difference indicates that although this pile group arrangement was symmetrical with the applied soil movement, a slight difference occurred to its geometry during the test setup, which lead to asymmetrical loadings to the piles. As a result, an additional bending moment is induced at one of the piles.

### Table 7.5 Summary of the response of piles A and B at $w_s = 130$ mm

<table>
<thead>
<tr>
<th>Pile response</th>
<th>Pile A</th>
<th>Pile B</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{+\text{max}}$ (Nmm)</td>
<td>5,307</td>
<td>7,681</td>
<td>45%</td>
</tr>
<tr>
<td>$S_{-\text{max}}$ (N)</td>
<td>-24.02</td>
<td>-20.54</td>
<td>17%</td>
</tr>
<tr>
<td>$r_{+\text{max}}$ (N/mm)</td>
<td>0.10</td>
<td>0.075</td>
<td>39%</td>
</tr>
<tr>
<td>Deflection at surface (mm)</td>
<td>64.56</td>
<td>64.55</td>
<td>0%</td>
</tr>
</tbody>
</table>

The piles deflection profiles (Figures 7.11(e) and 7.12(e)) show that both piles initially rotate at a depth of 420 mm ($w_s = 10 \sim 20$ mm). At a higher soil movement ($w_s = 30 \sim 130$ mm), the rotational point reduces to a shallower depth of approximately 320 mm. The rotation profiles (Figures 7.11(d) and 7.12(d)) show a constant rotation with the depth, indicating that the piles behave as rigid piles.
7.4.3 Test C_2-5_32_2×1_3d_270705

Another pile group test was conducted with the piles arranged in a column (2 × 1). Figures 7.13 and 7.14 show the responses of piles A and B, while a summary of the maximum value of the pile responses is listed in Table 7.6, at $w_s = 130$ mm. The pile deflection and the rotation profiles indicate that the piles behave as rigid piles. The locations of the $M_{\text{max}}$ for piles A and B are located at different locations of approximately 360 mm and 0 mm, respectively. The high magnitude of the $M_{\text{max}}$ at the soil surface on pile B shows that the pile cap and the pile arrangement have an effect on the bending moment, especially at the top section of the pile.

Table 7.6 Summary of the response of piles A and B at $w_s = 130$ mm

<table>
<thead>
<tr>
<th>Pile response</th>
<th>Pile A</th>
<th>Pile B</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{\text{+max}}$ (Nmm)</td>
<td>3,900</td>
<td>1,479</td>
<td>163.7%</td>
</tr>
<tr>
<td>$S_{\text{max}}$ (N)</td>
<td>-16.94</td>
<td>-19.34</td>
<td>14.2%</td>
</tr>
<tr>
<td>$r_{\text{max}}$ (N/mm)</td>
<td>0.06</td>
<td>0.15</td>
<td>150.0%</td>
</tr>
<tr>
<td>Deflection at surface (mm)</td>
<td>78.5</td>
<td>78.5</td>
<td>0%</td>
</tr>
</tbody>
</table>

The bending moment profiles of piles A and B, shown in Figures 7.13(a) and 7.14(a), respectively, indicate that these profiles are different in shapes. Thus, the $M_{\text{+max}}$ for piles A and B, differ by 163.7%.
Figure 7.11 Pile response of Test C_2-5_32_1×2_5d_A_300705 (pile A)
Figure 7.12 Pile response of Test C_2-5_32_1×2_5d_B_300705 (pile B)
Figure 7.13 Pile response of Test C_2-5_32_2\times1_3d__A_270705 (pile A)
Figure 7.14 Pile response of Test C_2-5_32_2×1_3d_B_270705 (pile B)
7.4.4 The effect of pile group arrangements

The effect of different pile group arrangements were investigated, comparing the single pile and two pile group tests. All piles had a diameter of 32 mm, with an $L_m/L_s$ ratio of 200/500. While each test was tested with a different $c_u$, the bending moment and the soil reaction were normalized with $c_u d^3$ and $c_u d$, respectively. Figure 7.15 shows the normalised bending moment profiles at $w_s = 60$ mm. The highest magnitude of the normalised bending moment ($M_{\text{max}}/c_u d^3$) is for the single pile tests, while the lowest is for pile B of the $2 \times 1$ pile group test. The difference between these $M_{\text{max}}/c_u d^3$ is significant. It is apparent, that at $w_s = 60$ mm, the bending moment profiles of piles A and B, respectively, for the $2 \times 1$ pile group, are similar in shape and magnitude. In contrast, at $w_s = 130$ mm, the difference on the bending moment profiles between the piles is significant (as discussed in Subsection 7.5.3). Figure 7.16 shows the normalized soil reaction profiles at $w_s = 60$ mm. On all tests, the maximum $r_{\text{max}}/c_u d$ in the moving and the stable layers are at depths of 60 mm (except for pile B in $2 \times 1$ pile group) and 360 mm, respectively. The $r_{\text{max}}/c_u d$ (in $L_m$ and $L_s$) of the single pile test is significantly higher than that noted in the pile group tests.

![Figure 7.15 Comparison of normalised bending moment profiles between pile groups](image-url)
Chapter 7: Single pile and pile group tests in clay

7.5 Observation of soil plugging

Clay plugging into the pile tip of the open-ended model pile was observed after the removal of the pile from the shear box at the completion of each test. The average soil plug measured up to approximately 100 mm, as shown in Figure 7.17.

Figure 7.17 Soil plug into the pile tip
7.6 Limitation of current testing with clay

Six tests were conducted on the piles in clay. These tests were more tedious and required a greater effort in the preparation of the clay than for those tests conducted in sand. The limitations of the tests and possible ways to address these limitations are listed below:

1) because of the clay sample used in the current tests was unconsolidated. However model tests with consolidated clay appear to show a better consistency (also repeatable with controlled $c_u$) in the test results (Pan, 1998; Bransby, 1994; Steward, 1992; Springman, 1989).

2) because of time required to prepare a large shear box test in clay, the size of the shear box was relatively small. This size prevented a bigger pile group from being tested, due to boundary effects between the pile and the shear box. However, according to Mcmanus and Kulhawy (1993a and b), the consolidation process for a clay sample in a shear box could take up to a month or more (varies with shear box size) to complete. Therefore, to improve the efficiency of the tests, the size of the shear box has to be carefully selected, with enough time allowed for the clay consolidation.

3) additional properties (strength and stiffness) testing was needed on the clay sample, especially, before and after each test (only vane shear test was carried out after the completion of each test in the current study). Also additional instrumentations in the clay sample could have provided a better interpretation of the test results. For example, a pressure transducer could have monitored the stress development within the shear box, while the soil movement was being applied to the piles.

4) the tests should be carried out with a controlled loading rate, particularly for the application of the soil movement, as clay is known to have time dependent effects.
7.7 Conclusions

Tests on single piles and pile groups subjected to a uniform soil (clay) movement profile were performed. The apparatus for testing with unconsolidated kaolin clay, were modified from the apparatus used for sand and previously described in Chapter 3. The strength \( (c_u) \) of the kaolin clay ranged from 2 to 15 kN/m². The current study had a number of limitations, which were indentified and addressed. The pile responses, indentified from the single pile and pile group tests, are summarised below:

**Single pile tests**

The normalised \( M_{\text{max}} \) \( (M_{\text{max}}/c_u d^2) \) increased with the pile diameter. In general, the bending moment, shear force, soil reaction, pile rotation profiles had a consistent shape, which remained unchanged, as \( w_s \) increased. The deflection profiles were different for each test. These profiles could be of translation and/or rotation at different rotation points. In all the tests, the rotation and the deflection profiles show that the pile behaved as a rigid pile.

**Pile group tests**

The tests conducted with the piles in a row arrangement \((1 \times 2)\) at 3d and 5d spacings, respectively, showed that the pile spacing has no significant effect on the pile response.

The bending moment profiles between piles A and B in the \(2 \times 1\) pile group had different shapes and magnitudes. The high magnitude of the \( M_{\text{max}} \) at the soil surface on pile B shows that the pile cap and the pile arrangement had an effect on the bending moment, particularly at the top section of the pile.

From the deflection and the rotation profiles, in all the pile group tests, it can be seen that both the piles (A and B) behaved as rigid piles.

The normalised \( M_{\text{max}} \) and \( r_{\text{max}} \) on the single pile test \((d = 32 \text{ mm})\) were higher than on the \((1 \times 2)\) and the \((2 \times 1)\) pile group tests.
In the current study, only six tests were conducted in clay, more tests are required, therefore, to both clarify and extend our knowledge of the pile behaviour.
Chapter 8: Simplified Solution to Analyse Rigid Pile Subjected to Lateral Soil Movement

8.0 Introduction

In this chapter, the attempt to simplify the problem of single piles subjected to lateral soil movement consisted of first dividing the pile into an unstable (or moving layer) and stable layer. Guo and Ghee (2003), Maugeri and Motta (1991), and Chen and Poulos (1997) had proposed solutions to solve these problems, which involved separating the piles into: 1) the upper portion of the pile embedded in the unstable layer of the soil, which is then subjected to lateral soil movement (referred to as a “passive” section); and 2) the lower section of the pile embedded in the stable layer, which provides lateral resistance to the lateral loading being transmitted from the upper pile portion (referred to as an “active” section.

A simplified solution proposed to analyse the rigid pile subjected to lateral soil movement, is presented in this chapter. The entire length of the pile, both the passive and active portions, were simplified with known pressure distributions along the pile. However, at this stage, the proposed simplified solution can only be used to analyse the pile subjected to rectangular soil movement. Additionally, the current solution cannot incorporate the axial load into the solution. Nevertheless, the simplified solution was much simpler to implement, and was a crucial component of a preliminary stage of the design. Four case studies presented later addressed the reliability of the proposed simplified solution.

8.1 Limiting pressure profile for sand

The magnitude maximum soil pressure, exerted on the pile at a given depth was the main input parameter of the simplified solution. This pressure determined the amount of the “disturbing” pressure the passive section could impose on the pile, and the amount of the “resisting” pressure that could be drawn from the active section of the pile to provide stability to the whole pile system. Many studies investigated pipelines, plat anchors and laterally loaded piles to identify the maximum soil pressure (also know as the limiting pressure) that could be exerted on these structures, The widely accepted limiting force
profiles for the laterally loaded piles are shown in Figure 8.1 (computed with the sand friction angle of $\phi = 38^\circ$). The current study, an assumption was made that the limiting pressure profile for the passive and the active sections of the piles were the same. In both of these sections, the relation proposed by Barton (1982) was used.

![Figure 8.1 Limiting pressure profiles for sand](image)

8.2 Soil movement to mobilise limiting pressure – with reference to pipe depth

Attewell et al (1986) reported many studies conducted on the behaviour of pipelines subjected to lateral soil movement, and found that, although the pipelines are embedded horizontally in the ground, the ultimate soil resistance can be calculated with the limiting pressure of the pile, as shown in Figure 8.1. It can be seen in Figure 8.2 that, at a particular depth, $z$, the limiting pressure, can be calculated for the pipelines or piles, with no difference in the limiting pressure.
In many instances, for a pile to be in a stable condition, the amount of limiting pressure mobilised, particularly, in the active section, is only a portion of the full limit. Thus the second input parameter for the simplified solution is the magnitude of the soil movement. Table 8.1 summarises the amount of soil movement required to mobilise the full limiting pressure from previous studies conducted on pipelines.

Table 8.1. Soil movement to mobilise limiting pressure on pipelines

<table>
<thead>
<tr>
<th>Reference</th>
<th>Soil</th>
<th>Displacement to mobilise, $P_u$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trautmann and O'Rourke (1985)</td>
<td>loose</td>
<td>0.13z</td>
</tr>
<tr>
<td></td>
<td>medium</td>
<td>0.08z</td>
</tr>
<tr>
<td></td>
<td>dense</td>
<td>0.03z</td>
</tr>
<tr>
<td>Nyman (1984)</td>
<td>loose</td>
<td>0.015z</td>
</tr>
<tr>
<td></td>
<td>dense</td>
<td>0.025z</td>
</tr>
<tr>
<td>Trautmann et al (1985)</td>
<td>loose ($\Phi = 31^\circ$)</td>
<td>0.005z</td>
</tr>
<tr>
<td></td>
<td>Dense ($\Phi = 44^\circ$)</td>
<td>0.015z</td>
</tr>
<tr>
<td>Rowe and Davis (1982)</td>
<td>loose ($\Phi = 32^\circ$)</td>
<td>0.003z*</td>
</tr>
<tr>
<td>Audibert and Nyman (1977)</td>
<td>loose</td>
<td>0.02z*</td>
</tr>
<tr>
<td></td>
<td>dense</td>
<td>0.015z*</td>
</tr>
<tr>
<td>Matyas and Davis (1983)</td>
<td>$\Phi = 30^\circ$ to $36^\circ$</td>
<td>0.0068z*</td>
</tr>
<tr>
<td>Casson (1984)</td>
<td>n/a</td>
<td>0.021z</td>
</tr>
</tbody>
</table>

*computed from the load deflection curve presented in the paper

+ $z$ used is depth to inverted level of pipe plus pipe diameter

^ $z$ used is depth to the inverted level of pipe
8.3 Soil movement to mobilise limiting pressure with reference to pile diameter

On a laterally loaded pile, it may be more appropriate to refer the soil movement in terms of the relative soil movement between the soil and the pile, especially as the pile moves, relative to the soil, as it deflects. Hence, there is no physical movement of the soil but the pile moves.

Regretably, few field tests available have studied the behaviour of a pile subjected to lateral soil movements; however, a larger number of field tests have investigated the behaviour of laterally loaded piles in sand. These tests offer the following recommendations in relation to the pile displacement needed to mobilise the limiting pressure:

- Williams et al (1988) reported that laterally loaded piles, embedded in calcareous soil, required a lateral pile displacement of 0.15d to mobilise the limiting pressure on the pile.

- A series of field tests on laterally loaded piles conducted by Reese et al. (1974) to investigate the load-deflection curve of the piles, or better know as the “p-y curve”, suggested that the limit of the pile deflection, needed to mobilise the limiting pressure, was \( y = 3.75d \% \).

- Briaud et al. (1983) conducted a review of a number of field tests on laterally loaded pile; the limiting pressure was identified as being reached at the pile displacement, \( y = 10d \% \).

Clearly, the amount of relative soil movement required to mobilise the full limiting pressure from the soil can be expressed in terms, of the function of the depth or as the function of the pile diameter. Given the above data, and assuming that there are no significant differences in limiting pressure between the passive and the active sections, in the simplified solution, the soil movement \( (w_s) \) of 0.15d was taken as the point where the limiting pressure was reached in the moving soil layer.
8.4 Three different failure modes of the rigid pile

The curvature of the pile, shown in Figure 8.3(a) was magnified to illustrate the direction of the bending of the rigid piles. At the low sliding depth ($L_m < L_s$), the pile deflection profile, Mode A, was comprised of only the rotation with a single curvature, and the maximum bending moment located slightly below the sliding depth. Further increases in $L_m$, to approximately equal $L_s$ ($L_m = L_s$), meant that the pile deflection profile, Mode B, comprised mainly of the rotation. In Mode B, the pile bends with double curvatures (Figure 8.3(b)). Finally, at $L_m > L_s$, the pile deflection profile, Mode C, comprised mainly of a transition with a slight rotation (Figure 8.3(c)). Figure 8.4 shows the different modes of failure observed and discussed in the previous chapters. The current experimental data comprised the results on Modes A and B only. Therefore, for comparative purposes, Mode C is taken from the available numerical analysis results (Chapter 6).

Figure 8.3. Different failure modes of rigid pile
8.5 Estimation of pressure profile in \( L_m \) (passive section)

The experimental model tests showed that the pressure distribution profile in the moving layer was parabolic in shape, and retained its shape (parabolic) for the different soil movement profiles (rectangular, triangular and arc) being applied to the pile. However, the magnitude of the pressure changed (linearly) with the magnitude of the soil movement \( (w_s) \) imposed on the pile (Figure 8.5(a)), where \( w_s \) was assumed to be constant for the entire \( L_m \) (rectangular soil movement). Nevertheless, the equation still held for the other soil movement profiles (triangular and parabolic). Thus, when \( w_s > 0.15d \) (Figure 8.5(c)), \( P_u \) remained constant. Hence, when \( w_s \) was large enough, the pressure distribution profile on the pile was the same in respect to any soil movement profile. The simplified pressure distribution profiles in \( L_m \), shown in Figures 8.5(a) and (b) correspond to \( L_m < L_s \) and \( L_m > L_s \), respectively. For the calculation, the limiting pressure, \( P_{us} \), was assumed to increase linearly with depth, up to \( w_s = 0.15d \) (Section 8.3).
8.6 Solution of pressure acting in \( L_s \) (active section)

The solution used to analyse the stable layer, \( L_s \), also known as the active section of passive piles, was based on the simplified pressure analysis proposed by Rutledge (1945), and then later by Prasad and Chari (1999) and Zhang et al (2005), as shown in Figure 8.6. The pressure distribution increased linearly with the applied pressure or load on the pile (Lavachev et al, 2003). Therefore, the load increment was assumed to be linearly increased, up to the ultimate load \( P_u \), when, after the \( P_u \) was reached, the pressure distribution of the rigid pile remained approximately constant; the deflection was expected to increase continuously to a higher magnitude (the pile in an unstable condition).

![Figure 8.5 Pressure distribution profiles in the moving layer (\( L_m \))](image1)

![Figure 8.6 Pressure profiles in the active section of the pile by Prasad and Chari (1999)](image2)
Chapter 8: Simplified solution to analyse rigid pile subjected to lateral soil movement

8.7 Pile rigidity

The current simplified solution was only applicable for the rigid pile (Figure 8.6). Hence, for the solution to be valid, the flexural rigidity of the pile had to be checked first. A number of researchers have proposed different expressions to identify the pile rigidity, namely:

- The pile rigidity can be checked by calculating the relative stiffness of the pile and the soil, $K_R = \frac{E_p I_p}{E_s L^4}$: 1) for a flexible pile, $K_R \leq 10^{-5}$; 2) for a medium flexible pile, $K_R \approx 10^{-3}$; and 3) for a rigid pile, $K_R \geq 10^{-1}$ (Poulos and Davis, 1980).

- The pile rigidity can be calculated as: 1) the pile is termed flexible when the depth to breadth to depth ratio, $D/B > 20$; and 2) the pile is termed rigid when $D/B < 6$ (Bierschwale et al, 1981)

- The pile rigidity can be calculated as: 1) $D/B < 0.05\left(\frac{E_c}{G^*}\right)^{\frac{1}{3}}$ for rigid piles; 2) $D/B > \left(\frac{E_c}{G^*}\right)^{\frac{2}{3}}$ for flexible piles; and 3) for intermediate stiff piles, $0.05\left(\frac{E_c}{G^*}\right)^{\frac{1}{3}} < D/B < \left(\frac{E_c}{G^*}\right)^{\frac{2}{3}}$, where $G^* = G_s \left(1 + \frac{3\nu}{4}\right)$ (Carter and Kulhawy, 1988)

- Kulhawy (1993) found from a number of field tests that, basically, the pile can be termed rigid when $D/B < 10$, and flexible when $D/B > 15$.

Any one of the expressions above can be used to validate that the pile is a rigid pile before the simplified solution is used for further analysis.
8.8 Solution flowchart

A spreadsheet was developed to facilitate the calculation procedure, shown in Figure 8.7, and describes the procedure for the initial check on the pile and the soil properties before further calculations are computed using the spreadsheet.

![Flowchart](image)

Figure 8.7 Flowchart for using the current simplified solution

8.9 Prediction on current single pile tests

The predictions, carried out using the proposed simplified solution on the model single pile tests, were reported in Chapter 4. Importantly, the behaviour of the model tests had a good agreement when compared with the numerical models reported in Chapter 6. The pile and the soil properties of Test 2-6_32_0k_600_170304 are summarised in Table 8.2.
Chapter 8: Simplified solution to analyse rigid pile subjected to lateral soil movement

These parameters were used directly as inputs into the spreadsheet of the simplified solution.

Table 8.2 Summary of pile and soil properties used in the simplified solution

<table>
<thead>
<tr>
<th>Pile external diameter (mm)</th>
<th>32</th>
<th>32</th>
<th>32</th>
<th>50</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moving layer, ( L_m ) (mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stable layer, ( L_s ) (mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil density (kN/m(^3))</td>
<td>200</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil friction angle, ( \phi )</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pile wall thickness (mm)</td>
<td>1.65</td>
<td></td>
<td></td>
<td>1.91</td>
<td></td>
</tr>
<tr>
<td>Pile stiffness, ( E_p I_p ) (Nmm(^2))</td>
<td>(1.28 \times 10^9)</td>
<td>(5.89 \times 10^9)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil movement, ( w_s ) (mm)</td>
<td>60</td>
<td>120</td>
<td>120</td>
<td>10.5</td>
<td>20.5</td>
</tr>
<tr>
<td>Soil movement profile</td>
<td>Rectangular</td>
<td>Triangular</td>
<td>Parabolic</td>
<td>Rectangular</td>
<td></td>
</tr>
<tr>
<td>Test reference</td>
<td>2-6_32_0k_600_170304</td>
<td>2-6_32_0k_600_240505_T</td>
<td>2-6_32_0k_600_210704_P</td>
<td>2-6_50_0k_600_110304</td>
<td></td>
</tr>
</tbody>
</table>

Figure 8.8 shows that the pile responses, in terms of the bending moment, shear force and soil reaction, for both the predicted and model tests, results. At \( w_s > 0.15d \) (or 4.8mm for 32mm pile), the simplified solution treated all soil movement profiles (rectangular, triangular or parabolic) equally, by imposing a constant pressure distribution profile in \( L_m \) (Figure 8.8(c)). The difference in the maximum bending moment between the predicted and the model tests results are 14 %, 49 % and 38 % for the rectangular, triangular and parabolic profiles, respectively. It is worth to noting that, the simplified solution over estimates the soil reaction (at up to 100 mm above the pile tip), by approximately four times. However, generally, the shapes of the pile responses are the same for the predicted and model test results. A summary of the critical pile responses is presented in Table 8.3.
Table 8.3 Summary of the critical pile responses with different soil movement profiles

<table>
<thead>
<tr>
<th>Pile response</th>
<th>Rectangular</th>
<th>Triangular</th>
<th>Parabolic</th>
<th>Predicted</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>in L_m</td>
<td>in L_s</td>
<td>in L_m</td>
<td>in L_s</td>
</tr>
<tr>
<td>$M_{\text{max}}$ (Nmm)</td>
<td>- 25,211</td>
<td>- 42,823</td>
<td>- 39,770</td>
<td>- 28,835</td>
</tr>
<tr>
<td>$S_{\text{max}}$ (N)</td>
<td>91</td>
<td>-93</td>
<td>174</td>
<td>-161</td>
</tr>
<tr>
<td>$f_{\text{max}}$ (N/mm)</td>
<td>-0.34</td>
<td>0.69</td>
<td>-0.69</td>
<td>1.26</td>
</tr>
</tbody>
</table>

Figure 8.8 Response of 32 mm diameter pile subject to rectangular profile soil movement ($L_{\text{m}}/L_{\text{s}} = 200/500$)
Figure 8.9 presents the pile responses of the 50 mm diameter pile at two magnitudes of soil movement: \( w_s = 10.5 \text{ mm} (0.21d); \) and \( w_s = 20.5 \text{ mm} (0.41d). \) At \( w_s = 10.5 \text{ mm}, \) or 0.21d, the limiting pressure within the moving layer \((L_m)\) is not fully mobilised. It is assumed that the \( w_s \) at the pile vicinity is half the total \( w_s \) (10.5 mm) applied at the boundary of the shear box. In the simplified solution, to cater for the partial mobilisation of the limiting pressure, a reduction factor of 0.7 or \((0.5 \times 10.5)/0.15d\) is used to calculate the pressure distribution profile in \( L_s \) (Figure 8.5).

As previously noted the magnitude of the soil movement needed to mobilise \( P_u \), can be difficult to determine, and very often depends on factors such as the relative density of the sand and depth of the pile (Section 8.2). The results from the model tests, reported in Chapter 4, indicate that the magnitudes of the soil movement required to mobilise \( P_u \) are 0.8d (20 mm), 0.6d (20 mm) and 0.4d (20 mm) for 25 mm, 32 mm and 50 mm piles, respectively.

In general, the shapes of the bending moment, the shear force and the soil reaction profiles are in good agreement with the predicted profiles. The magnitude and difference between the predicted and model test results are summarised in Table 8.4. For \( w_s = 10.5 \text{ mm}, \) the \( M_{\text{max}} \) and the maximum shear force compared very well with the predicted results, the difference being 19.8 % and 14.4 %, respectively. Also, as noted in the previous section, the simplified solution tends to over predict the soil reaction at the vicinity of the pile tip (Figure 8.9(c) and (e)).

### Table 8.4 Comparison between the predicted results from current solution and the measured results from the model test

<table>
<thead>
<tr>
<th>Pile response</th>
<th>( w_s = 10.5 \text{ mm} )</th>
<th>( w_s = 20.5 \text{ mm} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{\text{max}} ) (Nmm)</td>
<td>Measured</td>
<td>Predicted</td>
</tr>
<tr>
<td>42,816</td>
<td>35,733</td>
<td>19.8 %</td>
</tr>
<tr>
<td>( S_{\text{max}} ) (N)</td>
<td>159.36</td>
<td>143.69</td>
</tr>
<tr>
<td>( r_{\text{max}} ) (N/mm)</td>
<td>0.78</td>
<td>1.55</td>
</tr>
</tbody>
</table>
Figure 8.9 Response of 50 mm diameter pile subject to rectangular profile soil movement

\( \frac{L_m}{L_n} = 200/500 \)
In the following subsections (8.10.1 to 8.10.3), four case studies investigated the reliability of the current solution. They consisted of a model test and two large scale tests.

8.10.1 Cases 1 and 2 (Chen, 1994)

Chen (1994) conducted 1g model test on piles subjected to a triangular profile of lateral soil movement (a brief description of the experimental apparatus can be found in Chapter 2). The pile was made of aluminium tube, with an outer diameter, wall thickness and flexural stiffness of 25 mm, 1.2 mm and $E_{pl}l_p = 3.6 \times 10^8 \text{ Nmm}^2$, respectively. The total length of the pile was 1000 mm and the section of the pile embedded into the soil was 675 mm. The section of the pile in the moving ($L_m$) and stable layer ($L_s$) were 350 and 325 mm, respectively. The pile was instrumented with strain gauges at an interval of 100 mm, in calcareous sand originating from the Bass Strait, Australia. The maximum and the minimum dry densities were 14.8 kN/m$^3$ and 12.3 kN/m$^3$, respectively. The sand preparation involved ‘raining’ the sand (a similar procedure to that found in Chapter 3). The average density and the internal friction angle were 13.0 kN/m$^3$ and 40˚, respectively, while the soil movement profile was triangular, with the maximum soil movement ($w_s$) being recorded at the soil surface.

Two case studies (Cases 1 and 2) were carried on from the model tests reported by Chen (1994), which involved different sliding and stable layers, and soil movement. In Case 1, the sliding and the stable layers were 350 mm and 325 mm, respectively. The soil movement applied was $w_s = 65 \text{ mm}$. Table 8.5 shows a summary of the results from the simplified solution.
Table 8.5 Input parameters and comparison between results for Case 1

<table>
<thead>
<tr>
<th>Input parameter</th>
<th>Predicted</th>
<th>Chen (1994)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I</td>
<td>II</td>
</tr>
<tr>
<td>Moving layer, $L_m$ (mm)</td>
<td>225</td>
<td>250</td>
</tr>
<tr>
<td>Stable layer, $L_s$ (mm)</td>
<td>450</td>
<td>425</td>
</tr>
<tr>
<td>Soil movement, $w_s$ (mm)</td>
<td>65</td>
<td>65</td>
</tr>
<tr>
<td>$M_{max}$ (Nmm)</td>
<td>27,740</td>
<td>33,482</td>
</tr>
<tr>
<td>$S_{max}$ (N)</td>
<td>113.30</td>
<td>134.73</td>
</tr>
<tr>
<td>$r_{max}$ (N/mm)</td>
<td>1.44</td>
<td>1.90</td>
</tr>
</tbody>
</table>

Figure 8.10 presents the predicted pile responses in terms of bending moment, shear force and soil reaction profiles for Case 1. Different sets of the moving and table layers were used between each prediction. Figures 8.10(b) and (c) show the shear force and soil reaction profiles, respectively, that the measured maximum shear force at a depth of approximately 250 mm (the shear force is at a maximum value, while the soil reaction is at zero). As described in Chapter 3 (Subsection 3.3.2, on the sand displacement within the shear box), an investigation was attempted to identify the actual depth of the moving layer, however, it was found that the sliding depth consisted of a zone with finite thickness rather than a well defined interface. This zone was also found in the direct shear box tests, as noted by Atkinson and Bransby (1978).

In order to predict the model test results, four different moving layers ($L_m$) were used as inputs into the simplified solution. From the bending moment profiles (Figure 8.10(a)), it can be seen that the measured bending moment profile lies between the current predicted profiles (between $L_m$ of 250 mm and 300 mm).
In Case 2, the sliding and stable layers were 200 mm and 325 mm in depth, respectively. The soil movement applied was $w_s = 37$ mm. Figure 8.11 shows the pile responses from the predicted and the measured results. Two different sets of $L_m$ and $L_s$ were used in the simplified solution to obtain a good comparison. A summary of the input parameters and the critical pile responses are shown in Table 8.6. The measured bending moment profile lies between the predicted profiles. From the comparison and the shear force profiles (Figure 8.11(b), the location of the maximum shear force), it can be seen that, the actual $L_m$ is less than 200 mm.
Table 8.6 Input parameters and comparison between results for Case 2

<table>
<thead>
<tr>
<th>Input parameter</th>
<th>Predicted</th>
<th>Chen (1994)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I</td>
<td>II</td>
</tr>
<tr>
<td>Moving layer, $L_m$ (mm)</td>
<td>150</td>
<td>200</td>
</tr>
<tr>
<td>Stable layer, $L_s$ (mm)</td>
<td>375</td>
<td>325</td>
</tr>
<tr>
<td>Soil movement, $w_s$ (mm)</td>
<td>37</td>
<td>37</td>
</tr>
<tr>
<td>$M_{max}$ (Nmm)</td>
<td>5,448.22</td>
<td>9,687.92</td>
</tr>
<tr>
<td>$S_{max}$ (N)</td>
<td>29.21</td>
<td>49.92</td>
</tr>
<tr>
<td>$r_{max}$ (N/mm)</td>
<td>0.42</td>
<td>0.94</td>
</tr>
</tbody>
</table>

Figure 8.11 Case 2: The predicted and the measured pile responses for Chen (1994)
8.10.2 Case 3 (Tsuchiya et al, 2001)

Tsuchiya et al (2001) conducted a large-scale 1g test on piles subjected to triangular and trapezoidal profiles of lateral soil movement. A brief description of the experimental apparatus is described in Chapter 2. In this case study, the pile was made of 165.2 mm diameter steel pipe. The pile had a length and a wall thickness of 5 m and 3.7 mm, respectively. The calculated flexural stiffness of the pile was $E_pI_p = 1.27\text{ MN.m}^2$. The front and back of the pile was instrumented with strain gauges at intervals of 250 mm. The pile response was presented in terms of the bending strain.

Given the pile properties, the bending moment, the shear force and the soil reaction were calculated from the measured strain. The soil movement profile was triangular, with the maximum soil movement being recorded at the soil surface, $w_s = 32\text{ mm (0.194d)}$. The sand model used consisted of two layers: a 4 m thick upper layer of loose sand, with a relative density ranging from 10 % to 30 %; and a lower compacted sand layer, with a relative density of 60 % to 70 %. These relative densities correlated to the internal friction angles of $28^\circ$ (upper layer) and $36^\circ$ (lower layer), respectively (Schmertmann, 1975).

The current simplified solution took into account the two soil layers with different densities. In this case, the bottom 1,000 mm of the pile section was embedded into the denser sand layer. This layer, due to the current limitation of the simplified solution, was treated so that it had the same density as the upper layer ($D_r = 10\text{ to 30 % for entire }L_s$). This adjustment lead to both the $L_m$ and $L_s$ having the same density.

From the measured results, it was noted that the $w_s = 32\text{ mm (0.197d)}$, at the surface, was not sufficient to mobilise the limiting pressure for the whole depth of the moving layer ($L_m = 4000\text{ mm}$). Two different moving layers of 2,000 mm and 2,500 mm were used in the simplified solution, respectively. Correspondingly, by keeping the same pile length, the stable layers were 3000 mm and 2500 mm, respectively.
Table 8.7 Input parameters and results for Case 3

<table>
<thead>
<tr>
<th>Input parameter</th>
<th>Predicted</th>
<th>Tsuchiya et al (2001)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I</td>
<td>II</td>
</tr>
<tr>
<td>Moving layer, $L_m$  (mm)</td>
<td>2,000</td>
<td>2,500</td>
</tr>
<tr>
<td>Stable layer, $L_s$  (mm)</td>
<td>3,000</td>
<td>2,500</td>
</tr>
<tr>
<td>Soil movement, $w_s$ (mm)</td>
<td>32</td>
<td>32</td>
</tr>
<tr>
<td>$M_{max}$ (Nm)</td>
<td>5,063.82</td>
<td>8,129.08</td>
</tr>
<tr>
<td>$S_{max}$ (N)</td>
<td>-4,147.49</td>
<td>-7,694.97</td>
</tr>
<tr>
<td>$r_{max}$ (N/mm)</td>
<td>5.67</td>
<td>12.15</td>
</tr>
</tbody>
</table>

Figure 8.12 presents the pile responses of both the predicted and measured results. It appears that the bending moment from the Predicted I is in good agreement with the measured results, while the Predicted II over predicts the bending moment by 67%. The shear force profile also shows a good agreement between the measured and the predicted results with all three profiles having a similar shape. The soil reaction (at $L_m$) from the Predicted I and II are reduced to 0.24$P_u$, respectively, in order to take into account that only a fraction of the limiting pressure (at $L_m$) is fully mobilised at $w_s = 32$ mm (Section 8.5). The soil reaction profile from the Predicted I matches the measured result on both shape and magnitude. It is interesting to note that the zero soil reaction from the Predicted I and the measured results are at a depth of 2,500 mm, indicating the extent of the depth where the passive pressure (positive soil reaction) is acting on the pile, which is lower than the depth of the moving layer, report by the author ($L_m = 4,000$ mm). Thus, with a low magnitude of soil movement, together with a rather large sliding depth, only the upper section of the moving layer imposes a passive soil reaction to the pile.
Figure 8.12 Case 3: The predicted and the measured pile responses for Tsuchiya et al (2001)
Kalteziotis et al (1993) reported on a field test where the tubular piles were instrumented to study the response of the piles used to stabilise a sliding slope. The sliding soil layer consisted of marls, sandstones and conglomerates. Two rows of concrete bored piles, had a 1.0 m in diameter, 12 m in length, and spaced at 2.5 m intervals were installed to stabilise the sliding slope, as shown in Figure 8.13. The concrete bored piles were replaced with steel pipe piles, instrumented with vibrating wire strain gauges at every 1m along the entire length of the piles. These steel piles had an external diameter of 1.03 m, a wall thickness of 18 mm, and a flexural stiffness $E_p I_p$ of 1,540 MN.m².

Figure 8.13. Plan view of the sliding zone and location of the instrumented piles and inclinometers (after Kalteziotis et al, 1993)

Four inclinometers were installed in four different locations, to measure the soil movements at the uphill and downhill, are shown in Figures 8.14(a) and (b) respectively. The soil movement at the uphill was rather uniform with the depth, when compared with those at the downhill, which decreased linearly with the depth. The location of the soil movement shown in Figure 8.14(b) was recorded between the stabilising piles. Therefore, the soil movement should be smaller than the soil movements shown in Figure 8.14(a), which occurred far away from the piles.
In the present study, the maximum value of the uniform free field soil displacement, of 3.5 mm in the sliding soil layer and zero at a depth of 6 m below the soil surface, was assumed from the recorded inclinometer data. The following soil parameters were recorded from pressuremeter tests:

1. Moving layer, soil above the slip surface: \( P_{\text{moving}} = 0.9 \) MPa, \( E_s = 15 \) MPa; and
2. Stable layer, soil below the slip surface: \( P_{\text{stable}} = 3.2 \) MPa, \( E_s = 70 \) MPa.

![Figure 8.14. Lateral soil movement profile (after Kalteziotis et al, 1993)](image)

(a) N2: downhill

(b) G13: uphill

The measured soil movement profile changed from being uniformly distributed (rectangular) to being linearly varied with depth (triangular), as it approached the piles. The sliding depth was estimated from the soil movement profile of the uphill of the slope (soil movement without the influence of the piles). Also, from this soil movement profile, there was a “transition layer” at a depth of 2.8 m to 4.5 m, in which the soil movement was linearly decreased back to zero. In this case, the two possible depths of the moving layer, \( L_m \), were taken as 2.8 m and 4.5 m (both lie at the boundary of the transition layer).
Chapter 8: Simplified solution to analyse rigid pile subjected to lateral soil movement

Figure 8.15 compares the measured and the predicted pile responses (bending moment, shear force and soil reaction profiles). A summary of results are presented in Table 8.8.

![Graphs showing measured and predicted pile responses for Case 4](image)

Figure 8.15 Case 4: The predicted and the measured pile responses for Kaltezitis et al (1993)
Table 8.8 Input parameters and results for Case 4

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Moving layer, L_m (mm)</td>
<td>4,500</td>
<td>2,800</td>
<td>4,500</td>
</tr>
<tr>
<td>Stable layer, L_s (mm)</td>
<td>7,500</td>
<td>9,200</td>
<td>7,500</td>
</tr>
<tr>
<td>Soil movement, w_s (mm)</td>
<td>32</td>
<td>32</td>
<td>32</td>
</tr>
<tr>
<td>M_max (Nm)</td>
<td>133,019</td>
<td>97,959</td>
<td>136,686</td>
</tr>
<tr>
<td>S_max (N)</td>
<td>30,054</td>
<td>23,559</td>
<td>34,985</td>
</tr>
<tr>
<td>r_max (N/mm)</td>
<td>24.2</td>
<td>13.0</td>
<td>28.4</td>
</tr>
</tbody>
</table>

Generally, the predicted and the measured bending moment profiles show a good agreement in terms of the shape and the magnitude. However, the shear force profiles show a difference when compared to the measured results. It appears that the measured shear force fluctuated at depths of 2,000 mm and 8,000 mm. Since the authors did not present the soil reaction profile, the measured soil reaction profile shown in Figure 8.15(c) was calculated from the measured shear force profile (Figure 8.15(b)). The maximum shear forces (measured and predicted) appear to be at a depth of approximately 3,000 mm, which lies within the transition layer. On the other hand, the depth of maximum shear force, from the simplified solution, is always located at the interface between the moving and stable layers (depth = L_m). The differences between the predicted and the measured maximum soil reactions (positive) appear to be 17 % and 118 % for Predicted I and II, respectively.
8.11 Limitation of current solution

In the current study, the simplified solution did not address the following seven limitations:

1. the current simplified solution was limited to short (rigid) piles, as shown in the design flowchart (Figure 8.7)

2. The solution was only adequate to use for single piles with $L_m/L_s$ ratio of $< 0.5$. At this ratio, the $P_u$, from active section of the pile (Figure 8.6), was not fully mobilised. This also served as a safety factor to control excessive pile displacement, when the pressure on the pile reached the limiting pressure $P_u$.

3. although the solution was able to be used for single piles with $L_m/L_s > 0.5$, Figure 8.5(b), the solution was not verified, with a relevant case study, and so the active section of the pile needed further improvement with the redistribution of $P_u$ (in the pile).

4. the rectangular, triangular and parabolic soil movement profiles were treated equally in the solution. An adjustment can only be made to the thickness of moving layer ($L_m$) and soil movement ($w_s$) to cater for each soil movement profile. One possible improvement could be to introduce appropriate multiplication factors to cater for the different soil movement profiles.

5. only the piles in the uniform sand could be analysed, therefore no variation of soil density was permitted between $L_m$ and $L_s$.

6. the correlation between the magnitude of the soil movement and the percentage of the limiting pressure mobilised (Figure 8.5(b)) was empirical, being based on a limited numbers of field tests. Although the data were available from the model tests (Chapter 4), they may be less accurate when compared to the full-scale and centrifuge tests.

7. the solution appeared to over predict the soil reaction, notably, at the vicinity of the pile tip. One possible improvement was to revise the pressure distribution
8.12 Conclusions

The current study of the simplified solution proposed to predict the responses of the single piles subjected to lateral soil movements. The solution was based on the estimation of the pressure distribution profiles from the moving and stable soil layers.

A number of comparisons with the model tests (reported in Chapter 4) and the published test data shows that the simplified solution gave reasonably good predictions, especially, in relation to the shape of the different pile response profiles.

In all cases, the simplified solution tends to over predict the soil reaction profile in the vicinity of the pile tip. Since only a small portion of the pile is affected, generally, as being shown, it does not contribute to the overall prediction of the pile responses.

It was found that on the triangular and parabolic profiles, in many cases, the thickness of moving layer (Lₘ) reported in the published literatures can be different to the actual thickness of the moving layer (location where the shear force was at maximum) required by the simplified solution. In all case studies, the actual moving layer was smaller than that reported in the published literatures. The correct estimation of the actual moving layer had a great effect on the accuracy of the simplified solution.

Generally, the simplified solution gave a better prediction on the piles subjected to rectangular soil movement profile rather than the other profiles. The maximum difference of 50% on the $M_{\text{max}}$ was noted between the predicted and the measured results on the triangular soil movement profile. One possible improvement to the simplified solution would be to introduce multiplication factors to cater for the different soil movement profiles.
Chapter 9: Conclusions and Recommendations for Future Research Work

9.0 Introduction

The current research addressed, through experimental and numerical studies, the problem of axially loaded piles subjected to lateral soil movements. A number of major findings from these studies are summarised below.

9.1 Experimental study: piles in sand

A new testing apparatus was developed and used to conduct a series of tests on instrumented single piles and pile groups in sand. The measure strain responses from the piles were used to derive (numerical differentiation and integration) the bending moment, shear force, soil reaction, rotation and pile deflection profiles. Comparisons between the tests, related mainly to the bending moment, were made to identify the effect of various parameters on the pile responses.

9.1.1 Single pile tests

The following conclusions were drawn for the single piles subjected to three soil movement profiles (rectangular, triangular and arc). From the test results, the effects of a number of parameters on the pile response were identified. It has been indentified that, regardless of the soil movement profile, the pile diameter, density of sands, axial load and $L_m/L_s$ ratio, respectively, increased with the $M_{\text{max}}$. For the tests with low $L_m/L_s$ (= 200/500), the $M_{\text{max}}$ peaked at $w_s = 20$–$30$ mm, while, with a high $L_m/L_s$ (= 400/300), the $M_{\text{max}}$ increased with $w_s$, without reaching a definite peak value. For all the tests, the soil pressures, acting on the pile in the moving and the stable layers, respectively, did exceed the limiting pressure proposed by Barton (1982).
A comparison was made between the tests with the three soil movement profiles; the results were:

- at L_m/L_s = 200/500, the shape of the bending moment profiles were of a single curvature
- at L_m/L_s = 400/300, the shape of the bending moment profile was of a double curvature for the test with the rectangular soil movement, and single curvatures for the test with triangular and arc soil movements. Also, the maximum pile deflection on the test with the rectangular soil movement exceeded the magnitude of w_s, which, in turn, contributed to the negative soil reaction on the pile in L_m.

An unique linear relationship $M_{\text{max}} = \alpha S_{\text{max}}$, with $\alpha$ of 0.24, was found by fitting all the single pile tests results. This relationship was consistent with that proposed by Guo and Qin (2009).

9.1.2 Pile group tests

Only the rectangular soil movement profile was investigated for the pile group tests. Three pile group arrangements are used in the tests (1 × 2, 2 × 1 and 2 × 2). All piles in each pile group were connected together with a pile cap. A number of parameters were varied in the group tests, namely: the moving and the stable layers; the pile diameter; the axial load applied on the top of the pile cap; the pile group arrangement; and the pile group spacing. By varying these parameters, general trends were identified from the response of the two instrumented piles within the pile group.

The findings from a comparison between the three pile group arrangements showed that:

- the pile spacing has a significant effect on the pile the pile response on the 2 × 1 and the 2 × 2 pile groups, however the spacing has a negligible effect on the 1 × 2 pile group
- in relation to the 2 × 1 and 2 × 2 pile group, a significant negative bending moment was found on the soil surface, owing to the restraint provided by the pile cap. This restraint also caused the $M_{\text{max}}$ to increase, while the restraint increased with the axial load applied on the pile group, and the pile spacing
in relation to the 2 × 2 pile group only, the front piles (leading piles facing the moving soil) caused a ‘shadowing’ effect on the back piles. The result was significantly more soil pressure (from the moving soil) being imposed on the front piles compare to the back piles.

The following three findings were identified, regardless of the pile group arrangements:

- the $M_{\text{max}}$ increased with the increase of either the $L_m/L_s$ ratio or the pile diameter
- in general, the maximum soil pressure acting on the pile in the moving layer or the stable layer of the soil, did not exceed the limiting pressure proposed by Barton (1982)
- an unique linear relationship $M_{\text{max}} = \alpha S_{\text{max}}$, with $\alpha$ of 0.22 to 0.24, was found by fitting all the pile group test results. This relationship was consistent with that proposed by Guo and Qin (2009).

9.2 Numerical analysis: single piles in sand

FLAC$^{3D}$ program was successfully used to predict the results of the single model tests that are associated with large deformations due to moving soil. In the FLAC$^{3D}$ program, using the large strain mode, the soil was modelled as elements or zones which were allowed to deform with the change in the surrounding stresses. The parametric study, performed on a standard model, indicated that the Young's modulus, friction angle and dilation angle of the sand had a significant effect on the magnitude of the maximum bending moment on the pile.

Subsequent analyses indicated good agreements between the FLAC$^{3D}$ predictions and the experimental results, especially in terms of the shape of the bending moment, shear force, soil reaction, pile rotation, and pile deflection profiles. However, in all analyses, the FLAC$^{3D}$ under predicted the maximum bending moment obtained from the model tests.
Two main factors contributed to the results of the FLAC$^{3D}$ predictions, namely:

- the effect of the initial pile verticality, and the way the axial load was applied on the pile head, caused different degrees of p-delta effects on the pile
- the apparent scale effect on the tests, with a shallow moving layer, for instance, the test with $L_m \leq 200$ mm. When the sand dilation angle was taken as equal to the sand friction angle ($\phi = \psi$), the predicted results were closer to the experimental results than when using with $\phi = \psi - 30^\circ$.

In the model tests, only two ratios of $L_m/L_s$ of 200/500 and 400/300 were investigated. A study, using the FLAC$^{3D}$, investigated a wider range of $L_m/L_s$ on the pile behaviour. Results from this study showed that:

- the deflection mode of the pile changed with an increase in $L_m/L_s$
- the maximum bending moment peaks at the $L_m/L_s$ ratio were close to unity
- the sand dilation angle had the greatest effect on the maximum positive bending moment at $L_m/L_s < 1.0$, and had a negligible effect at $L_m/L_s > 2.0$
- the relation $M_{max} = (0.14 \sim 0.25)S_{max}$ fitted well with the FLAC$^{3D}$ results with different $L_m/L_s$, which was also found to be consistent with that proposed by Guo and Qin (2009).

9.3 Experimental study: piles in clay

Tests on the single piles and pile groups subjected to uniform soil (clay) movement profile were performed. The apparatus used for testing with the unconsolidated kaolin clay were modified from the apparatus used for sand. The strength ($c_u$) of the kaolin clay ranged in between 2 and 15 kN/m$^2$. A number of the indentified limitations were addressed. The pile responses indentified from the tests are summarised in Subsections 9.3.1 and 9.3.2.

9.3.1 Single pile tests

The normalised $M_{max}$ ($M_{max}/c_u d^2$) increased with the pile diameter. In general, the bending moment, shear force, soil reaction, and pile rotation profiles had a consistent shape, which remains unchanged, as $w_s$ increased. The deflection profiles were
different between each test, being either translation and/or rotation at different rotation points. In all the tests, the rotation and the deflection profiles showed that the pile behaved as a rigid pile.

9.3.2 Pile group tests

The tests conducted with the piles in a row arrangement (1 × 2) at 3d and 5d spacings showed no significant effect on the pile responses. The bending moment profiles between piles A and B in the (2 × 1) pile group were different in shape and magnitude. The high magnitude of the $M_{\text{max}}$ at the soil surface on pile B showed that the pile cap and the pile arrangement had an effect on the bending moment, particularly at the top section of the pile. In all pile group tests, it can be seen from the deflection and the rotation profiles that both the piles (A and B) behaved as rigid piles. The normalised $M_{\text{max}}$ and $r_{\text{max}}$ on the single pile test ($d = 32\,\text{mm}$) were higher than for the (1 × 2) and the (2 × 1) pile group tests.

Only six tests were conducted with clay; more tests are required to further the investigation of pile behaviour.

9.4 Simplified solution to analyse rigid pile subjected to lateral soil movement

A simplified solution was proposed to help predict the responses of single piles subjected to lateral soil movements. The solution was based on the estimation of the pressure distribution profiles, from the moving and the stable soil layers, respectively, acting on the pile. This pressure profile was, in turn, integrated to the 1st and 2nd order to derive the shear force and the bending moment profiles, respectively.

A number of case studies carried out to predict the results from the model tests and published tests showed that the simplified solution gave reasonably good predictions, especially, the shape of the different pile response profiles.

The simplified solution gave a better prediction for the piles subjected to the rectangular soil movement profile rather than other soil movement profiles. The maximum difference of 50 % on the $M_{\text{max}}$ was noted between the predicted and the
measured results on the triangular soil movement profile. At this study, the simplified solution was only intended for piles subjected to the rectangular soil movement profile. However, further research could address the possible improvements to the simplified solution by introducing multiplication factors to cater for the different soil movement profiles.

9.5 Recommendations for future research work

Along the way of this research work, many limitations and possible improvements have been tabulated in each chapter. Furthermore, there are a number of areas where are not carried in the current research that permit future work to be carried out. Some suggestions for future work are presented under the following Subsections 9.5.1 to 9.5.5

9.5.1 Experimental apparatus

It would be useful to:

1) develop a ‘mini’ inclinometer to help measure rotation and pile deflection profiles
2) place soil pressure cells at different location within the shear box to help measure the stresses development at different locations within the shear box
3) have a comprehensive investigation to assess the way sand particles move within the shear box, in order to relate better the actual sand movement to the pile response, and help in interpreting the test results
4) add more strain gauge channels to instrument more piles in a group (currently only instrumented piles are used in each pile group test)
5) use full-bridge strain gauge (effectively acting as a load cell) connections to measure the axial stress distribution along the pile.
Chapter 9: Conclusions and recommendations for future research work

9.5.2 Single pile tests

It would be useful if:

1) additional ratios of the moving against the stable layers could be tested (in the presented study only $L_m/L_s = 200/500$ and $400/300$ were tested)
2) additional levels of axial load were applied on the piles (in the present study $Q = 0$ N, 196 N, 294 N and 589 N)
3) an investigation was undertaken into the effect of the pile head condition (fixed head)
4) tests were undertaken on piles embedded in the moving and the stable layers with different sand density.

9.5.3 Pile group tests

It would be useful if:

1) the number of the pile group arrangements were increased
2) the pile groups were subjected to triangular and parabolic soil movement profiles.
3) the pile groups had a free-head
4) additional levels of axial load were applied on the pile groups (in the present study the maximum of $Q = 294$ N per pile was applied on the pile cap).

9.5.4 Numerical analysis

It would be good if:

1) the modelling of single piles was subjected to triangular and parabolic soil movement
2) the modelling of pile groups was undertaken
3) other stress dependent constitutive models on sand were investigated
4) the modelling of piles in clay were undertaken.
9.5.5 Simplified solution

It would be good if:

1) additional case studies, particularly, on different soil movement profiles was undertaken
2) established modification factors were introduced to cater for different soil movement profiles
3) improvements were made to the current solution for single piles in clay
4) the solution to cater for \( \frac{L_{np}}{L_s} > 0.5 \) was extended.

9.6 Concluding remarks

The current thesis presents a study of the responses of axially loaded piles subjected to lateral soil movements. The importance of expanding our knowledge of structural piles and how they react to axial load and soil movements cannot be underestimated for the purpose of analysis and design. Indeed from the current study, a greater understanding into the mechanism of the pile behaviour has been obtained. The laboratory model tests on single piles and pile groups identified the way the piles respond to a number of parameters. The numerical study helped to predict the results from the model tests, illustrating the ability of the three-dimensional finite difference program to model the behaviour of the piles. In the absence of a three-dimensional program, a new simplified solution was developed; it proved useful for a preliminary analysis of single piles subjected to rectangular soil movement. Nevertheless, additional research is required to explore, experimentally, other areas (detailed above), to improve the numerical analysis; and to improve the analytical solution. At this stage, the numerical analysis, although relatively time consuming to implement, is able to provide a complete detailed analysis.
References


Brinch Hansen, J. (1961). The ultimate resistance of rigid piles against transversal forces, Bulletin No. 12, Danish Geotechnical Institute, Copenhagen, Denmark, pp. 5-9.


Appendix A

Sand properties
Appendix A1

Direct shear box test on sand

Direct Shear Box Test for Sand
(Australian Standard: 1289.6.2.2-1998)

<table>
<thead>
<tr>
<th>Test Number</th>
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<tr>
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<tr>
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</tr>
<tr>
<td>Breadth (mm)</td>
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<tr>
<td>Area (mm²)</td>
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<td>Normal Stress (kPa)</td>
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</tr>
</tbody>
</table>

![Graphs showing shear stress vs. shear displacement for different normal stresses, vertical displacement vs. shear displacement for different loads, and shear stress vs. normal stress.]
Appendix A2

Oedometer test on sand

Confined Compression Test on Sand with Oedometer

Diameter of ring (mm) 63.5
Initial dry unit weight (approximately) 14.9 kN/m³
Area, (m²) 0.00316692
Final dry unit weight (approximately) 17.7 kN/m³
Initial thickness (mm) 20
Poisson's ratio 0.3

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<th>Load (kg)</th>
<th>Vertical Stress (kPa)</th>
<th>Stress Increment (kPa)</th>
<th>Settlement (mm)</th>
<th>Volumetric Strain (%)</th>
<th>Strain Increment (%)</th>
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<th>Young's Modulus (*)</th>
<th>Shear Modulus (MPa)</th>
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<td>0</td>
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<td>1.135</td>
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Note: "E_s" is calculated from constrained secant modulus with Poisson's ratio of sand taken as 0.3
Appendix A3

Relative density tests on sand

**Determination of the Relative Density of the Sand Bed**

Soil compaction and density tests - Determination of the minimum and maximum dry density of a cohesionless material - Standard Method (Australian Standard, AS 1289.5.5.1-1998)

Conducted on 19/09/03

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<td>Mass of mold + water (g)</td>
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<td>11048</td>
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Samples obtained from sand rainer

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Estimated friction angle (°) 38 (Schmertmann, 1978) Estimated friction angle (°) 38 (Schmertmann, 1978)

Soil Moisture Content Check after Testing (29/09/03)

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Appendix B

Single pile test results
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<th>L₁⁺, L₂⁺ (mm)</th>
<th>Nₛₑ (N-MM)</th>
<th>Depth (mm)</th>
<th>Stable skin (N)</th>
<th>Earring layer (N)</th>
<th>Pile deflection (mm)</th>
<th>Soil Movement (mm)</th>
<th>Passive (sliding layer) (mm)</th>
<th>Active (slide layer) (mm)</th>
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Note: It should be noted that the soil movement (Δ₁, Δ₂) applied during each individual test pertains to the purpose of comparison and presentation in Chapter 4 of the thesis, where necessary. The pile responses are interpolated back to that at w₉ = 10, 20, 30, 40 and 60 mm, respectively.
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<td>Soil Moving Profile</td>
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<tr>
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<td>Axial load (N)</td>
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![Diagram of test setup](image1)

![Graph of Soil Movement](image2)

![Graph of Rotation](image3)

![Graph of Shear Force](image4)

![Graph of Deflection](image5)

![Graph of Bending Moment](image6)

![Graph of Soil Reaction](image7)
Test Number: 2-5_50_20k_600_081105

Date: 08/11/2005  
Falling Height (mm): 600

Pile-Head condition: Free-head  
Soil Moving Profile: Rectangular  
Diameter (mm): 50

Moving layer, L_m (mm): 200  
Stable layer, L_s (mm): 500  
Axial load (N): 196

![Soil Movement Diagram]

![Bending Moment Diagram]

![Rotation Diagram]

![Shear Force Diagram]

![Displacement Diagram]

![Soil Reaction Diagram]
Test Number: 4-3_32_0k_600_200904

Date: 20/09/2004  
Falling Height (mm): 600

Pile-Head condition: Free-head  
Soil Moving Profile: Rectangular

Moving layer, \( L_m \) (mm): 400  
Stable layer, \( L_s \) (mm): 300  
Axial load (N): 0

![Graphs showing Soil Movement, Rotation, Shear Force, Soil Reaction, and Bending Moment vs. Depth.](4-3_32_0k_600_200904.xls)
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</tr>
<tr>
<td>Pile-Head condition</td>
<td>Free-head</td>
</tr>
<tr>
<td>Soil Moving Profile</td>
<td>Rectangular</td>
</tr>
<tr>
<td>Diameter (mm)</td>
<td>32</td>
</tr>
<tr>
<td>Moving layer, L_m (mm)</td>
<td>200</td>
</tr>
<tr>
<td>Stable layer, L_s (mm)</td>
<td>500</td>
</tr>
<tr>
<td>Axial load (N)</td>
<td>294</td>
</tr>
</tbody>
</table>

![Diagram of soil movement and load distribution](image)

- **Rotation**
- **Shear Force (N)**
- **Deflection (mm)**
- **Soil Reaction (N/mm)**
- **Bending Moment (Nmm)**

2-5_32_30k_600_260304.xls
Test Number 4-3_32_30k_600_240904

Date 26/03/2004

Pile-Head condition Free-head

Soil Moving Profile Rectangular

Diameter (mm) 32

Moving layer, L_m (mm) 400

Stable layer, L_s (mm) 300

Axial load (N) 294
Test Number: 4-3_32_60k_600_040205
Date: 04/02/2005
Falling Height (mm): 600
Pile-Head condition: Free-head
Soil Moving Profile: Rectangular
Diameter (mm): 32
Moving layer, $L_m$ (mm): 400
Stable layer, $L_s$ (mm): 300
Axial load (N): 588

Graphs showing:
- Soil Movement
- Bending Moment (Nmm)
- Rotation
- Shear Force (N)
- Deflection (mm)
- Soil Reaction (N/mm)
- Bending Moment (Nmm)
- Deflection (mm)
- Soil Reaction (N/mm)
Test Number: 2-5_50_0k_600_Rep_100605
Date: 10/06/2005
Falling Height (mm): 600
Pile-Head condition: Free-head
Soil Moving Profile: Rectangular
Diameter (mm): 50
Moving layer, L_m (mm): 200
Stable layer, L_s (mm): 500
Axial load (N): 0

Soil Movement

Bending Moment (Nmm)

Rotation

Shear Force (N)

Displacement (mm)

Soil Reaction (N/mm)
<table>
<thead>
<tr>
<th>Test Number</th>
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</tr>
</thead>
<tbody>
<tr>
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<td>Pile-Head condition</td>
<td>Free-head</td>
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<tr>
<td>Soil Moving Profile</td>
<td>Rectangular</td>
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<tr>
<td>Diameter (mm)</td>
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</tr>
<tr>
<td>Moving layer, L_m (mm)</td>
<td>400</td>
</tr>
<tr>
<td>Stable layer, L_s (mm)</td>
<td>300</td>
</tr>
<tr>
<td>Axial load (N)</td>
<td>0</td>
</tr>
</tbody>
</table>

This document contains various graphs showing soil movement, displacement, rotation, shear force, bending moment, and soil reaction as a function of depth. Each graph is labeled with different data points indicating various conditions and measurements. The graphs are designed to illustrate the behavior of soil under different load conditions and falling heights. The data is referenced from Excel file 4-3_50_0k_600_100904.xls.
Test Number: 2-5_50_30k_600_050404

Date: 05/04/2004

Falling Height (mm): 600

Pile-Head condition: Free-head

Soil Moving Profile: Rectangular

Diameter (mm): 50

Moving layer, Lm (mm): 200

Stable layer, Ls (mm): 500

Axial load (N): 294
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<td>Diameter (mm)</td>
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<td>Stable layer, L_s (mm)</td>
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<tr>
<td>Axial load (N)</td>
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</tr>
</tbody>
</table>

![Diagram of pile and soil movement](image)

![Graph of Bending moment (Nmm) vs Depth (mm)](image)

![Graph of Rotation vs Depth (mm)](image)

![Graph of Shear Force (N) vs Depth (mm)](image)

![Graph of Displacement (mm) vs Depth (mm)](image)

![Graph of Soil Reaction (N/mm) vs Depth (mm)](image)
Test Number: 4-3_50_60k_600_140504
Date: 14/05/2004
Falling Height (mm): 600
Pile-Head condition: Free-head
Soil Moving Profile: Rectangular
Diameter (mm): 50
Moving layer, \( L_m \) (mm): 400
Stable layer, \( L_s \) (mm): 300
Axial load (N): 588

- Bending Moment (Nmm)
- Rotation
- Shear Force (N)
- Displacement (mm)
- Soil Reaction (N/mm)
<table>
<thead>
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<tbody>
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<td>Free-head</td>
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<td>Soil Moving Profile</td>
<td>Rectangular</td>
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<tr>
<td>Diameter (mm)</td>
<td>50</td>
</tr>
<tr>
<td>Moving layer, $L_m$ (mm)</td>
<td>200</td>
</tr>
<tr>
<td>Stable layer, $L_s$ (mm)</td>
<td>500</td>
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<tr>
<td>Axial load (N)</td>
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![Graphs showing various soil movements and forces](image-url)
<table>
<thead>
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<th>Test Number</th>
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<tbody>
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<td>Date</td>
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<td>Pile-Head condition</td>
<td>Free-head</td>
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<td>Soil Moving Profile</td>
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<tr>
<td>Diameter (mm)</td>
<td>50</td>
</tr>
<tr>
<td>Moving layer, $L_m$ (mm)</td>
<td>200</td>
</tr>
<tr>
<td>Stable layer, $L_s$ (mm)</td>
<td>500</td>
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<tr>
<td>Axial load (N)</td>
<td>0</td>
</tr>
</tbody>
</table>

![Graphs showing Soil Movement, Bending Moment, Rotation, Shear Force, Displacement, and Soil Reaction](image-url)
Test Number: 2-5_25_0k_400_240805

Date: 24/08/2005

Falling Height (mm): 600

Pile-Head condition: Free-head

Soil Moving Profile: Rectangular

Diameter (mm): 25

Moving layer, $L_m$ (mm): 200

Stable layer, $L_s$ (mm): 500

Axial load (N): 0

Bending Moment (Nmm) vs. Depth (mm)

Rotation vs. Depth (mm)

Shear Force (N) vs. Depth (mm)

Deflection (mm) vs. Depth (mm)

Soil Reaction (N/mm) vs. Depth (mm)
**Test Number**: 2-5_32_0k_400_220404  
**Date**: 22/04/2004  
**Falling Height (mm)**: 400  
**Pile-Head condition**: Free-head  
**Soil Moving Profile**: Rectangular  
**Diameter (mm)**: 32  
**Moving layer (mm)**: 200  
**Stable layer (mm)**: 500  
**Axial load (N)**: 0

---

**Soil Movement**

**Rotation**

**Shear Force (N)**

**Displacement (mm)**

**Bending Moment (Nmm)**

---

**Soil Reaction (N/mm)**
<table>
<thead>
<tr>
<th>Test Number</th>
<th>2-5_50_0k_400_080404</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date</td>
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<tr>
<td>Falling Height (mm)</td>
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</tr>
<tr>
<td>Pile-Head condition</td>
<td>Free-head</td>
</tr>
<tr>
<td>Soil Moving Profile</td>
<td>Rectangular</td>
</tr>
<tr>
<td>Diameter (mm)</td>
<td>50</td>
</tr>
<tr>
<td>Moving layer, L_m (mm)</td>
<td>200</td>
</tr>
<tr>
<td>Stable layer, L_s (mm)</td>
<td>500</td>
</tr>
<tr>
<td>Axial load (N)</td>
<td>0</td>
</tr>
</tbody>
</table>

![Graph of Soil Movement](image)

![Graph of Bending Moment (Nmm)](image)

![Graph of Rotation](image)

![Graph of Shear Force (N)](image)

![Graph of Displacement (mm)](image)

![Graph of Soil Reaction (N/mm)](image)
Test Number: 2-5_32_0k_200_100104
Date: 22/04/2004
Falling Height (mm): 200
Pile-Head condition: Free-head
Soil Moving Profile: Rectangular
Diameter (mm): 32
Moving layer, $L_m$ (mm): 200
Stable layer, $L_s$ (mm): 500
Axial load (N): 0

- **Rotation**
  - Depth (mm)
  - Rotation values from 0 to 0.006

- **Soil Reaction (N/mm)**
  - Depth (mm)
  - Values range from -60 to 60

- **Shear Force (N)**
  - Depth (mm)
  - Values range from -100 to 100

- **Displacement (mm)**
  - Depth (mm)
  - Values range from -5000 to 15000

- **Bending Moment (Nmm)**
  - Depth (mm)
  - Values range from 0 to 15000

- **W_s**
  - Values: 11, 28, 40, 55, 72, 99

---

2-5_32_0k_200_100104.xls  367
<table>
<thead>
<tr>
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<tbody>
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<tr>
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<td>Pile-Head condition</td>
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<tr>
<td>Soil Moving Profile</td>
<td>Rectangular</td>
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<tr>
<td>Diameter (mm)</td>
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<td>Moving layer, $L_m$ (mm)</td>
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<tr>
<td>Stable layer, $L_s$ (mm)</td>
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</tr>
<tr>
<td>Axial load (N)</td>
<td>0</td>
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</tbody>
</table>

**Graphs:**
- Bending moment (Nmm) vs. Depth (mm)
- Rotation vs. Depth (mm)
- Displacement (mm) vs. Depth (mm)
- Shear Force (N) vs. Depth (mm)
- Soil Reaction (N/mm) vs. Depth (mm)
- Bending moment (Nmm) vs. Depth (mm)
## Test summary for single pile in sand (triangular soil movement profile)

<table>
<thead>
<tr>
<th>Pile-head condition</th>
<th>Soil Moving Profile</th>
<th>Pile dia (mm)</th>
<th>Lm-Ls (mm)</th>
<th>Mmax (KN.mm)</th>
<th>Depth (mm)</th>
<th>Shear force (N)</th>
<th>Pile deflection (mm)</th>
<th>Soil Movement</th>
<th>Passive (sliding layer)</th>
<th>Active (stable layer)</th>
<th>File name</th>
<th>Constants</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free-head</td>
<td>Triangular (dense sand)</td>
<td>32</td>
<td>0</td>
<td>200/500</td>
<td>42.82</td>
<td>360</td>
<td>-174.31</td>
<td>5.8</td>
<td>0-40</td>
<td>-1.9</td>
<td>8.85 0-60</td>
<td>BM starts increasing after 50 mm displacement</td>
</tr>
<tr>
<td></td>
<td></td>
<td>400/300</td>
<td>14.98</td>
<td>460</td>
<td>80.79</td>
<td>460</td>
<td>-181.57</td>
<td>6.99</td>
<td>0-40</td>
<td>1.9</td>
<td>0-60</td>
<td>BM starts increasing after 50 mm displacement</td>
</tr>
<tr>
<td></td>
<td></td>
<td>120/400</td>
<td>123.09</td>
<td>460</td>
<td>203.55</td>
<td>460</td>
<td>-215.17</td>
<td>121.85</td>
<td>200</td>
<td>1.2</td>
<td>0-60</td>
<td>BM starts increasing after 50 mm displacement</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200/500</td>
<td>29.21</td>
<td>360</td>
<td>175.97</td>
<td>200</td>
<td>-210.46</td>
<td>162.56</td>
<td>1.2</td>
<td>2.1</td>
<td>0-60</td>
<td>BM starts increasing after 50 mm displacement</td>
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<tr>
<td></td>
<td></td>
<td>400/300</td>
<td>36.74</td>
<td>460</td>
<td>140.63</td>
<td>460</td>
<td>-180.72</td>
<td>7.17</td>
<td>0-40</td>
<td>-1.1</td>
<td>0-60</td>
<td>BM stops increasing after 70 mm displacement</td>
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<tr>
<td></td>
<td></td>
<td>300/500</td>
<td>170.98</td>
<td>460</td>
<td>460.44</td>
<td>460</td>
<td>-255.17</td>
<td>200.56</td>
<td>200</td>
<td>1.2</td>
<td>200</td>
<td>BM stops increasing after 50 mm displacement</td>
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<tr>
<td></td>
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<td>0</td>
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<td>200</td>
<td>BM stops increasing after 50 mm displacement</td>
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<tr>
<td></td>
<td></td>
<td>400/300</td>
<td>82.94</td>
<td>460</td>
<td>281.36</td>
<td>460</td>
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<td>460</td>
<td>261.6</td>
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<td>400/300</td>
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<td>460</td>
<td>818.85</td>
<td>460</td>
<td>-918.32</td>
<td>16.94</td>
<td>10-160</td>
<td>1.34</td>
<td>0-160</td>
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<td>2.3</td>
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<td>130/300</td>
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<td>460</td>
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<td>50.95</td>
<td>10-140</td>
<td>1.34</td>
<td>0-140</td>
<td>BM stops increasing after 50 mm displacement</td>
</tr>
</tbody>
</table>

**Notes:**
- **Lm-Ls:** Distance between the pile head and the reference point.
- **Mmax:** Maximum bending moment.
- **Depth:** Depth of the pile.
- **Shear force:** Shear force acting on the pile.
- **Pile deflection:** Pile deflection.
- **Soil Movement:** Movement of the soil around the pile.
- **Passive (sliding layer):** Passive resistance to sliding.
- **Active (stable layer):** Active resistance to the pile.
- **File name:** Identification of the test.
- **Constants:** Conditions during the test.

**Legend:**
- BM: Bending moment.
- Opposite side: Indicates the side where the observation is made.
**Test Number**: 2-5_32_0k_600_200505_T

- **Date**: 20/05/2005
- **Falling Height (mm)**: 600
- **Pile-Head condition**: Free-head
- **Soil Moving Profile**: Triangular
- **Diameter (mm)**: 32
- **Moving layer, Lm (mm)**: 200
- **Stable layer, Ls (mm)**: 500
- **Axial load (N)**: 0

**Graphs**:
- Bending Moment (Nmm)
- Rotation
- Shear Force (N)
- Displacement (mm)
- Soil Reaction (N/mm)

**File**: 2-5_32_0k_600_200505_T.xls
Test Number: 4-3_32_0k_600_071005_T

Date: 07/10/2005

Falling Height (mm): 600

Pile-Head condition: Free-head

Soil Moving Profile: Triangular

Diameter (mm): 32

Moving layer, Lm (mm): 400

Stable layer, Ls (mm): 300

Axial load (N): 0
Test Number: 2-5_32_30k_600_240505_T
Date: 24/05/2005
Falling Height (mm): 600
Pile-Head condition: Free-head
Soil Moving Profile: Triangular
Diameter (mm): 32
Moving layer, L_m (mm): 200
Stable layer, L_s (mm): 500
Axial load (N): 294

![Graphs of soil movement and force distributions](image-url)
<table>
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<tbody>
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<tr>
<td>Soil Moving Profile</td>
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<td>32</td>
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<td>Moving layer, L_m (mm)</td>
<td>400</td>
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<td>Stable layer, L_s (mm)</td>
<td>300</td>
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<tr>
<td>Axial load (N)</td>
<td>294</td>
</tr>
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</table>

![Graphs showing various soil properties](image-url)
Test Number 2-5_50_0k_600_250505_T
Date 25/05/2005 Falling Height (mm) 600
Pile-Head condition Free-head Soil Moving Profile Triangular Diameter (mm) 50
Moving layer, \( L_m \) (mm) 200 Stable layer, \( L_s \) (mm) 500 Axial load (N) 0

![Graphs of Soil Movement, Rotation, Shear Force, Displacement, and Soil Reaction](2-5_50_0k_600_250505_T.xls)
<table>
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<td>Soil Moving Profile</td>
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<tr>
<td>Diameter (mm)</td>
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<td>Axial load (N)</td>
<td>294</td>
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![Graphs showing various measurements and plots relating to soil mechanics.]
<table>
<thead>
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<th>4-3_50_30k_600_080904_T</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date</td>
<td>08/09/2004</td>
</tr>
<tr>
<td>Falling Height (mm)</td>
<td>600</td>
</tr>
<tr>
<td>Pile-Head condition</td>
<td>Free-head</td>
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<tr>
<td>Soil Moving Profile</td>
<td>Triangular</td>
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<tr>
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<td>50</td>
</tr>
<tr>
<td>Moving layer, $L_m$ (mm)</td>
<td>400</td>
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<tr>
<td>Stable layer, $L_s$ (mm)</td>
<td>300</td>
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<tr>
<td>Axial load (N)</td>
<td>294</td>
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</tbody>
</table>

![Graphs showing soil movement, rotation, shear force, and displacement](4-3_50_30k_600_080904_T.xls)
<table>
<thead>
<tr>
<th>Pile-head condition</th>
<th>Soil Moving Profile</th>
<th>Pile dia (mm)</th>
<th>t (kg)</th>
<th>Lₐₗ₃</th>
<th>Mₘₐₓ (KN-mm)</th>
<th>Depth (mm)</th>
<th>Shear force (N)</th>
<th>Pile deflection (mm)</th>
<th>Soil Movement</th>
<th>Passive (sliding layer)</th>
<th>Active (stable layer)</th>
<th>File name</th>
<th>Comments</th>
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Test summary for single pile in sand (arc soil movement profile)
Test Number: 2-5_32-ok_600_210704_P
Date: 21/07/2004
Falling Height (mm): 600
Pile-Head condition: Free-head
Soil Moving Profile: Arc
Diameter (mm): 32
Moving layer, Lm (mm): 200
Stable layer, Ls (mm): 500
Axial load (N): 0

![Graphs showing Soil Movement, Rotation, Shear Force, Displacement, and Bending Moment](image-url)
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**Soil Reaction (N/mm)**

**Displacement (mm)**

**Shear Force (N)**

**Rotation**

**Bending Moment (Nmm)**

**Soil Reaction (N/mm)**

4-3_32_0k_600_220704_P.xls
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![Graphs showing various engineering data](2-5_32_30k_600_250306_P.xls)
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Date: 17/10/2005
Pile-Head condition: Free-head
Soil Moving Profile: Arc
Falling Height (mm): 600
Diameter (mm): 32
Moving layer, Lm (mm): 400
Stable layer, Ls (mm): 300
Axial load (N): 294
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![Graphs showing soil movement, bending moment, rotation, shear force, and soil reaction.](image-url)
Test Number: 4-3_50_0k_600_250804_P
Date: 25/08/2004
Falling Height (mm): 600
Pile-Head condition: Free-head
Soil Moving Profile: Arc
Diameter (mm): 50
Moving layer, Lm (mm): 400
Stable layer, Ls (mm): 300
Axial load (N): 0

Soil Reaction (N/mm)

Bending Moment (Nmm)

Displacement (mm)

Shear Force (N)

Rotation
Test Number 2-5_50_30k_600_150705_P
Date 15/07/2005 Falling Height (mm) 600
Pile-Head condition Free-head Soil Moving Profile Arc Diameter (mm) 50
Moving layer, Lm (mm) 200 Stable layer, Ls (mm) 500 Axial load (N) 294

![Diagram of soil profile with labeled parameters]

- Bending Moment (Nmm)
- Shear Force (N)
- Rotation
- Displacement (mm)
- Soil Reaction (N/mm)

2-5_50_30k_600_150705_P.xls
Test Number: 4-3_50_30k_600_181005_P
Date: 18/10/2005
Falling Height (mm): 600
Pile-Head condition: Free-head
Soil Moving Profile: Arc
Diameter (mm): 50
Moving layer, Lm (mm): 400
Stable layer, Ls (mm): 300
Axial load (N): 294

Soil Movement

Soil Reaction (N/mm)

Bending Moment (Nmm)

Shear Force (N)

Rotation

Displacement (mm)

Bending Moment (Nmm)

Shear Force (N)

Displacement (mm)
Appendix C

Pile group test results
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### Notes:
- BM = Baseline Moment
- (+ve) = Higher than Baseline
- (-ve) = Lower than Baseline
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**Graphs:**
- **Axial Load**
- **Bending Moment (Nmm)**
- **Rotation**
- **Shear Force (N)**
- **Displacement (mm)**
- **Soil Reaction (N/mm)**
Test Number: 2-5_32_60k_600_1x2_3d_y_A_290405

Arrangement: 1x2

Date: 29/04/2005
Falling Height (mm): 600
Spacing: 3x(dia)

Pile-Head condition: Group
Soil Moving Profile: Rectangular
Diameter (mm): 32

Moving layer, \( L_m \) (mm): 200
Stable layer, \( L_s \) (mm): 500
Axial load (N): 588

Axial Load

Bending Moment (Nmm)

Section x-x

Soil Reaction (N/mm)

Displacement (mm)

Shear Force (N)

Rotation

Bending Moment (Nmm)

Shear Force (N)

Displacement (mm)

Soil Reaction (N/mm)
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<td>Depth (mm)</td>
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Date 15/02/2005
Falling Height (mm) 600
Spacing 3x(dia)
Arrangement 1x2
Pile-Head condition Group Soil Moving Profile Rectangular Diameter (mm) 32
Moving layer, Lm (mm) 400 Stable layer, Ls (mm) 300 Axial load (N) 0

Axial Load

Rotation

Displacement (mm)

Soil Reaction (N/mm)

Bending Moment (Nmm)

Shear Force (N)

Section x-x

4-3_32_0k_600_1x2_3d_y_B_150205.xls
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<td>Axial load (N)</td>
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<td>Axial load (N)</td>
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</table>

**Diagram:**
- **Axial Load**
- **Bending Moment (Nmm)**
- **Rotation**
- **Shear Force (N)**
- **Displacement (mm)**
- **Shear Force (N)**
- **Soil Reaction (N/mm)**

**Graphs:**
- Axial Load vs. Depth
- Bending Moment vs. Depth
- Rotation vs. Depth
- Shear Force vs. Depth
- Displacement vs. Depth
- Soil Reaction vs. Depth

**Legend:**
- Soil Movement
- Axial Load
- Bending Moment
- Rotation
- Shear Force
- Displacement
- Soil Reaction

**Section x-x**

**Output:**

4-3_32_60k_600_1x2_3d_y_A_180205.xls
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<td>Axial load (N)</td>
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**Axial Load**

**Bending Moment (Nmm)**

**Displacement (mm)**

**Shear Force (N)**

**Rotation**

**Soil Reaction (N/mm)**

**Section x-x**
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Axial Load

![Axial Load Diagram](image)

Bending Moment (Nmm)

![Bending Moment Graph](image)

Rotation

![Rotation Graph](image)

Shear Force (N)

![Shear Force Graph](image)

Displacement (mm)

![Displacement Graph](image)

Soil Reaction (N/mm)

![Soil Reaction Graph](image)
Test Number 2-5_32_0k_600_2x1_3d_x_B_141204
Date 14/12/2004
Falling Height (mm) 600
Spacing 3x(dia)
Pile-Head condition Group Soil Moving Profile Rectangular Diameter (mm) 32
Moving layer, L_m (mm) 200 Stable layer, L_s (mm) 500 Axial load (N) 0

Axial Load

Bending Moment (Nmm)

Rotation

Shear Force (N)

Displacement (mm)

Soil Reaction (N/mm)
Test Number: 2-5_32_60k_600_2x1_3d_x_A_010205
Date: 01/02/2005
Falling Height (mm): 600
Spacing: 3x(dia)
Arrangement: 2x1

Pile-Head condition:
- Group: Soil Moving Profile
- Rectangular
- Diameter (mm): 32

Moving layer, L_m (mm): 200
Stable layer, L_s (mm): 500
Axial load (N): 588

Axial Load

Bending Moment (Nmm)

Depth (mm)

Rotation

Shear Force (N)

Displacement (mm)

Soil Reaction (N/mm)

Depth (mm)
Test Number 2-5_32_60k_600_2x1_3d_x_B_010205

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Moving layer, \( L_m \) (mm) 200
Stable layer, \( L_s \) (mm) 500

Shear Force (N)

Displacement (mm)

Rotation

Axial Load

Bending Moment (Nmm)

Shear Force (N)

Soil Reaction (N/mm)

Section x-x
Test Number: 4-3_32_0k_600_2x1_3d_x_A_100205

Arrangement: 2x1

Date: 10/02/2005

Falling Height (mm): 600

Spacing: 3×(dia)

Pile-Head condition: Group

Soil Moving Profile: Rectangular

Diameter (mm): 32

Moving layer, L_m (mm): 400

Stable layer, L_s (mm): 300

Axial load (N): 0

Displacement (mm): 0 - 80

Rotation: 0 - 0.08

Depth (mm): 0 - 800

Shear Force (N): -10000 - 40000

Bending Moment (Nmm): -10000 - 40000

Soil Reaction (N/mm): -200 - 200

Section x-x

Axial Load

w_s, soil movement

L_m

L_s

A

X

X

4-3_32_0k_600_2x1_3d_x_A_100205.xls
Test Number: 4-3_32_0k_600_2x1_3d_x_B_100205
Arrangement: 2x1
Date: 10/02/2005
Falling Height (mm): 600
Spacing: 3x(dia)

Pile-Head condition: Group Soil Moving Profile: Rectangular
Diameter (mm): 32
Moving layer, Lm (mm): 400
Stable layer, Ls (mm): 300
Axial load (N): 0

Axial Load

Bending Moment (Nmm)

Rotation

Shear Force (N)

Displacement (mm)

Soil Reaction (N/mm)
Test Number: 4-3_32_60k_600_2x1_3d_x_A_251005
Arrangement: 2x1
Date: 25/10/2005
Falling Height (mm): 600
Spacing: 3x(dia)
Pile-Head condition: Group
Soil Moving Profile: Rectangular
Diameter (mm): 32
Moving layer, Lm (mm): 400
Stable layer, Ls (mm): 300
Axial load (N): 588

Axial Load

Bending Moment (Nmm)

Rotation

Shear Force (N)

Displacement (mm)

Soil Reaction (N/mm)
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![Axial Load Diagram](image1)

![Bending Moment Diagram](image2)

![Rotation Diagram](image3)

![Shear Force Diagram](image4)

![Displacement Diagram](image5)

![Soil Reaction Diagram](image6)
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<td>Axial load (N)</td>
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**Axial Load**

**Displacement (mm)**

**Rotation**

**Bending Moment (Nmm)**

**Shear Force (N)**

**Soil Reaction (N/mm)**

**Section x-x**
Test Number: 2-5_32_0k_600_2x2_3d_B_291004
Arrangement: 2x2
Date: 29/10/2004
Falling Height (mm): 600
Spacing: 3x(dia)

Pile-Head condition:
Group: Soil Moving Profile
Rectangular
Diameter (mm): 32

Moving layer, Lm (mm): 200
Stable layer, Ls (mm): 500
Axial load (N): 0

Axial Load

Bending Moment (Nmm)

Rotation

Shear Force (N)

Displacement (mm)

Soil Reaction (N/mm)
Test Number: 2-5_32_120k_600_2x2_3d_B_031104
Arrangement: 2x2
Date: 03/11/2004
Falling Height (mm): 600
Spacing: 3x(dia)
Pile-Head condition: Group
Soil Moving Profile: Rectangular
Diameter (mm): 32
Moving layer, L_m (mm): 200
Stable layer, L_s (mm): 500
Axial load (N): 1177

Axial Load

Section x-x

Bending Moment (Nmm)

Rotation

Shear Force (N)

Displacement (mm)

Soil Reaction (N/mm)

2-5_32_120k_600_2x2_3d_B_031104.xls
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<tr>
<td>Axial load (N)</td>
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**Axial Load Diagram**

- Depth (mm): 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110, 120, 130, 140
- Rotation: 0, 0.004, 0.008, 0.012
- Displacement (mm): 0, 2, 4, 6, 8, 10, 12
- Soil Reaction (N/mm): -240, -200, -160, -120, -80, -40, 0, 40, 80
- Shear Force (N): -60000, -40000, -20000, 0, 20000
- Bending Moment (Nmm): -60000, -40000, -20000, 0, 20000

**Section x-x**

- L_m
- L_s
- A

---

2-5_50_120k_600_2x2_3d_A_101104.xls

411
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Diagram:
- Axial Load diagram
- Bending Moment diagram
- Rotation diagram
- Shear Force diagram
- Displacement diagram
- Soil Reaction diagram

**Section x-x**
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Axial Load

Bending Moment (Nmm)

Rotation

Shear Force (N)

Displacement (mm)

Soil Reaction (N/mm)
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### Diagrams

- **Axial Load**
  - Section x-x
  - Lm, L, A
  - w, soil movement

- **Bending Moment (Nmm)**
  - Depth (mm)
  - Lm, L, A

- **Rotation**
  - Depth (mm)
  - Lm, L, A

- **Shear Force (N)**
  - Depth (mm)
  - Lm, L, A

- **Displacement (mm)**
  - Depth (mm)
  - Lm, L, A

- **Soil Reaction (N/mm)**
  - Depth (mm)
  - Lm, L, A

---

2-5_32_120k_600_2x2_5d_A_161104.xls  415
Test Number: 2-5_32_120k_600_2x2_5d_B_161104
Arrangement: 2x2

Date: 16/11/2004
Falling Height (mm): 600
Spacing: 5x(dia)

Pile-Head condition:
- Group
- Soil Moving Profile: Rectangular
- Diameter (mm): 32

Moving layer, L_m (mm): 200
Stable layer, L_s (mm): 500
Axial load (N): 1177

Axial Load

Bending Moment (Nmm)

Rotation

Shear Force (N)

Displacement (mm)

Soil Reaction (N/mm)

Section x-x
Test Number: 2-5_50_0k_600_2x2_5d_A_191104  
Arrangement: 2x2  

Date: 19/11/2004  
Falling Height (mm): 600  
Spacing: 5x(dia)  

Pile-Head condition: Group  
Soil Moving Profile: Rectangular  
Diameter (mm): 50  
Moving layer, $L_m$ (mm): 200  
Stable layer, $L_s$ (mm): 500  
Axial load (N): 0

Axial Load

$w_s$, soil movement

Section x-x

Bending Moment (Nmm)

Displacement (mm)

Rotation

Shear Force (N)

Soil Reaction (N/mm)

Depth (mm)

Axial Load

$w_s$, soil movement

Section x-x

Bending Moment (Nmm)

Displacement (mm)

Rotation

Shear Force (N)

Soil Reaction (N/mm)

Depth (mm)
Test Number: 2-5_50_0k_600_2x2_5d_B_191104  
Arrangement: 2x2  
Date: 19/11/2004  
Falling Height (mm): 600  
Spacing: 5x(dia)  
Pile-Head condition: Group  
Soil Moving Profile: Rectangular  
Diameter (mm): 50  
Moving layer, L_m (mm): 200  
Stable layer, L_s (mm): 500  
Axial load (N): 0

Axial Load

Section x-x

Bending Moment (Nmm)

Rotation

Shear Force (N)

Displacement (mm)

Soil Reaction (N/mm)
Test Number 2-5_50_120k_600_2x2_5d_A_241104
Arrangement 2x2

Date 24/11/2005
Falling Height (mm) 600
Spacing 5x(dia)

Pile-Head condition Group Soil Moving Profile Rectangular Diameter (mm) 50
Moving layer, Lm (mm) 200 Stable layer, Ls (mm) 500 Axial load (N) 1177

Axial Load

Bending Moment (Nmm)

Rotation

Shear Force (N)

Displacement (mm)

Soil Reaction (N/mm)
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<tr>
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Test Number: 4-3_32_0k_600_2x2_3d_A_220205
Arrangement: 2x2

Date: 22/02/2005
Falling Height (mm): 600
Spacing: 5x(dia)

Pile-Head condition:
Group: Shore,
Soil Moving Profile: Rectangular
Diameter (mm): 32

Moving layer, Lm (mm): 400
Stable layer, Ls (mm): 300
Axial load (N): 0

Axial Load

Bending Moment (Nmm)

Section x-x

Rotation

Shear Force (N)

Displacement (mm)

Soil Reaction (N/mm)
Test Number: 4-3_32_0k_600_2x2_3d_B_220205  
Arrangement: 2x1

Date: 22/02/2005  
Falling Height (mm): 600  
Spacing: 3x(dia)

Pile-Head condition: Group Soil Moving Profile  
Rectangular  
Diameter (mm): 32

Moving layer, L_m (mm): 400  
Stable layer, L_s (mm): 300  
Axial load (N): 0

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Test Number: 4-3_32_120k_600_2x2_3d_A_260205
Date: 26/02/2005
Falling Height (mm): 600
Spacing: 3x(dia)

Pile-Head condition: Group Soil Moving Profile Rectangular Diameter (mm): 32
Moving layer, L_m (mm): 400 Stable layer, L_s (mm): 300 Axial load (N): 1177
Test Number 4-3_32_120k_600_2x2_3d_B_260205  
Test Number 4-3_32_120k_600_2x2_3d_B_260205 Arrangement 2x2  
Date 26/02/2005  
Falling Height (mm) 600  
Spacing 3x(dia)  
Pile-Head condition Group Soil Moving Profile Rectangular Diameter (mm) 32  
Moving layer, Lm (mm) 400  
Stable layer, Ls (mm) 300  
Axial load (N) 120

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Axial Load  
Section x-x
Test Number: 4-3_32_0k_600_2x2_5d_A_020305
Date: 02/03/2005
Falling Height (mm): 600
Spacing: 5x(dia)

Pile-Head condition: Group
Soil Moving Profile: Rectangular
Diameter (mm): 32

Moving layer, Lm (mm): 400
Stable layer, Ls (mm): 300
Axial load (N): 0

Axial Load

Bending Moment (Nmm)

Rotation

Shear Force (N)

Displacement (mm)

Soil Reaction (N/mm)
Test Number 4-3_32_0k_600_2x2_5d_B_020305
Arrangement 2x1
Date 02/03/2005
Falling Height (mm) 600
Spacing 3x(dia)
Pile-Head condition Group Soil Moving Profile Rectangular Diameter (mm) 32
Moving layer, Lm (mm) 400
Stable layer, Ls (mm) 300
Axial load (N) 0

Axial Load

Bending Moment (Nmm)

Rotation

Shear Force (N)

Displacement (mm)

Soil Reaction (N/mm)
**Table:**

<table>
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<td>Pile-Head condition</td>
<td>Group Soil Moving Profile Rectangular Diameter (mm)</td>
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<td>Moving layer, Lm (mm)</td>
<td>400 Stable layer, Ls (mm)</td>
<td>300 Axial load (N)</td>
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</tbody>
</table>

**Diagram:**

- Axial Load
- Bending Moment (Nmm)
- Rotation
- Shear Force (N)
- Displacement (mm)
- Soil Reaction (N/mm)

**Section x-x**
Appendix D

Standard model input datafile for FLAC$^3$D
title
The response of piles in sands undergoing lateral movement (uniform Profile, 4-3.50_600_0k)

;---------------------------------------------------------------------Generate zones---------------------------------------------
gen zone radc p0 (0,0,0) p1 (0.5,0,0) p2 (0,-0.7) p3 (0.5,0,5,0) &
p4 (0.5,0,-0.7) p5 (0.5,0.5,-0.7) p6 (0.5,0.5,0) p7 (0.5,0.5,-0.7) &
p8 (0.025,0,0) p9 (0,0.025,0) p10 (0.025,0,-0.7) p11 (0,0.025,-0.7) &
size 1 28 4 5 ratio 1 1 1.5

gen zone radc p0 (0,0,-0.7) p1 (0.5,0,-0.7) p2 (0,0,-0.8) p3 (0.5,-0.7) &
p4 (0.5,0,-0.8) p5 (0.5,-0.8) p6 (0.5,0,-0.7) p7 (0.5,0.5,-0.8) &
p8 (0.025,0,-0.7) p9 (0,0.025, -0.7) p10 (0.025,0,-0.8) p11 (0,0.025,-0.8) &
size 6 4 4 5 ratio 1 1 1.5 fill

gen zone reflect dd 270 dip 90
group sand

;---------------------------------------------------------------------Generate interfaces--------------------------------------
interface 1 face range cylin end1 (0,0,0) end2 (0,0,-0.7001) radius 0.02501 &
cylin end1 (0,0,0) end2 (0,0,-0.7001) radius 0.0230 not

interface 2 face range cylin end1 (0,0,-0.6999) end2 (0,0,-0.7001) &
radius 0.02501

;---------------------------------------------------------------------Generate cylinder pile-------------------------------------
gen zone cshell p0 (0.1,1.300) p1 (0.025,0.1,1.300) p2 (0,0,0.1) p3 (0.025,1.300) &
p4 (0.025,0,0.1) p5 (0.025,0.1) p8 (0.0231,0,1.300) p9 (0,0.0231,1.300) &
p10 (0.0231,0.1) p11 (0,0.0231,0.1) &
size 1 48 4
gen zone reflect dd 270 dip 90 range z 0.1 1.300
group pile range z 0.1 1.300
ini add -0.800 range group pile
attach face range z -0.7001 -0.6999

;---------------------------------------------------------------------Soil, pile and interface properties----------------------
; Cohesion = 0, (Es=0.572MN/m2, poisson=0.3, Gs=0.220MN/m2)
model mohr range group sand
prop bulk 0.477e6 shear 0.220e6 coh 0 fric 38 dil 8 ten 0 range group sand
model elast range group pile
prop bulk 0.477e6 she range group pile
interface 1 prop kn 1e8 ks 1e8 fric 28 dil 0 coh 0 ten 0
interface 2 prop kn 1e8 ks 1e8 fric 28 dil 0 coh 0 ten 0

;---------------------------------------------------------------------Initial density of soil-------------------------------
;(Density of soil = 1627 kg/m3)
ini dens 1627 range group sand
ini dens 1627 range group pile

;---------------------------------------------------------------------Boundary and fixity condition--------------------------
;fix z range z -0.0001 0.0001 ;(Free at the top surface)
fix z range z -0.8001 -0.7999 ;Fix z-dir at bottom surface
fix x range x -0.5001 -0.4999 ;Fix x-dir at left surface
fix x range x 0.4999 0.5001 ;Fix x-dir at right surface
fix y range y -0.0001 0.0001 ;Fix y-dir at symmetrical surface
fix y range y 0.4999 0.5001 ;Fix y-dir at outer surface

;---------------------------------------------------------------------Set gravity---------------------------------------------
;(Gravity=10m/s2)
set gravity 0 0 -10

;---------------------------------------------------------------------Initial stresses----------------------------------------
;Ko=1-sin(28)=0.38
ini szz 0. grad 0 0 16270 range z -0.8 0
ini sxx 0. grad 0 0 6182 range z -0.8 0
ini syy 0. grad 0 0 6182 range z -0.8 0
set large

Histories
hist unbal
pl hist 1
save geh1_1
step 2000
save geh1_2

Pile properties
model elas range group pile
prop bulk 6.86e10 shear 2.63e10 range group pile
ini dens 2700 range group pile
step 40000
save geh1_3

Apply initial x-velocity
ini xvel 1e-7 range x -0.5001 -0.4999 z -0.4001 1.0000
ini xvel 1e-7 range x 0.4999 0.5001 z -0.4001 1.0000
ini state 0
ini xdis 0.0 ydis 0.0 zdis 0.0

Histories
hist n 100000
hist gp xdisp (-0.0231,0,0) ; Pile displacement (Fr); hist 2
hist gp xdisp (-0.0231,0,0) ; Pile displacement (Fr); hist 3
hist gp xdisp (-0.0231,0,0) ; Pile displacement (Fr); hist 4
hist gp xdisp (-0.0231,0,0) ; Pile displacement (Fr); hist 5
hist gp xdisp (-0.0231,0,0) ; Pile displacement (Fr); hist 6
hist gp xdisp (-0.0231,0,0) ; Pile displacement (Fr); hist 7
hist gp xdisp (-0.0231,0,0) ; Pile displacement (Fr); hist 8
hist gp xdisp (-0.0231,0,0) ; Pile displacement (Fr); hist 9
save geh1_4
pl his 1
step 1500000
save geh1_5

pl his 1
step 1500000
save geh1_5