Piles as settlement reducers in raft

Kambala, Siva Nagi Reddy

2010

Kambala, S. N. R. (2010). Piles as settlement reducers in raft. Doctoral thesis, Nanyang Technological University, Singapore.

<https://hdl.handle.net/10356/42530>

<https://doi.org/10.32657/10356/42530>

**NANYANG
TECHNOLOGICAL
UNIVERSITY**

PILES AS SETTLEMENT REDUCES IN RAFT

KAMBALA SIVA NAGI REDDY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
2010

PILES AS SETTLEMENT REDUCES IN RAFT

KAMBALA SIVA NAGI REDDY

School of Civil and Environmental Engineering

A thesis submitted to the Nanyang Technological University
in fulfillment of the requirement for the degree of
Doctor of Philosophy

2010

ACKNOWLEDGEMENTS

Firstly, I would like to express my heartfelt gratitude and sincere appreciation to my supervisor Dr. Teh Cee Ing. He was extremely patient and always willing to help. His unfailing interest, perception and appreciation of events of technical importance were extremely helpful for the completion of this work and will not be forgotten. I am indebted to my supervisor for his kindness and help throughout this research.

I would like to express my sincere thanks to Nanyang Technological University for giving me an opportunity to do research, for providing scholarship during initial period of my study and for funding the equipment required for research.

I wish to thank, Mr. Christopher Chia, Mr. Eugene, Mr. Tey and other geotechnical laboratory staff for their kindness and invaluable assistance during my experimental work.

I also wish to thank all my fellow graduate colleagues and friends for their encouragement and moral support. I wish to thank Dr. Damu, Dr. Kalyan, Dr. Murthy Dr. Kulkarni, Dr. Ramamohan and Mrs. Hemalatha for their suggestions, ideas and help during the course of my study. I would like to utilise this opportunity to thank all my teachers who taught me many courses.

I wish to thank Aecom Singapore management, Mr. Peter Lee, Dr. Ganesan and Mr. Khalid for their encouragement, help and considerations during thesis writing. Many thanks to all my Aecom colleagues for their encouragement and understanding during final stages of my research work.

I wish to express my love and gratitude to my parents Mr. Sambi Reddy and Mrs. Sujatha for their unfailing love, understanding and encouragement in all my endeavours. I wish to thank my uncle Mr. Sankar Reddy for his encouragement and support during my studies. I wish to thank my friend Mr. Appa Rao for his encouragement and love. I thank my brother Umamaheswar Reddy for his love and support.

Finally, many special thanks to my wife Nagamani for her unconditional love, understanding and encouragement in completing this work.

TABLE OF CONTENTS

ACKNOWLEDGEMENTS	i
ABSTRACT	ii
TABLE OF CONTENTS	iii
LIST OF TABLES	viii
LIST OF FIGURES	xii
LIST OF PHOTOGRAPHS	xxiii
LIST OF SYMBOLS	xxiv
Chapter 1 Introduction	1
1.1 Background	1
1.2 Objective of Present Research	3
1.3 Scope of Present Work	4
1.4 Outline of Thesis	4
Chapter 2 Literature Review	6
2.1 Introduction	6
2.2 Concept of Piled Rafts	6
2.2.1 <i>Favourable and unfavourable site conditions for piled rafts</i>	6
2.2.2 <i>Different categories of piled raft foundations</i>	7
2.3 Piled raft Design Philosophies	8
2.3.1 <i>Conventional approach</i>	8
2.3.2 <i>Creep piling approach</i>	10
2.3.3 <i>Differential settlement control approach</i>	11
2.4 Field Studies of Piled-Raft Foundations	14
2.5 Experimental Studies	30
2.6 Analysis of Piled Rafts	36
2.6.1 <i>Simplified calculation methods</i>	37
2.6.2 <i>Approximate computer-based methods</i>	40
2.6.3 <i>More rigorous computer-based methods</i>	51
2.7 Summary	63

Chapter 3	Experimental Set-up and Test Procedures	66
3.1	Introduction	66
3.2	Foundation Soil	67
3.3	Model Rafts	67
3.4	Model Piles	69
3.5	Equipment and Measurement System used in Model Tests	71
3.5.1	<i>Slurry mixer and consolidation tank</i>	72
3.5.2	<i>Loading system</i>	74
3.5.3	<i>Instrumentation systems</i>	76
3.5.4	<i>Data acquisition system</i>	82
3.5.5	<i>Other ancillary equipments</i>	83
3.6	Typical Procedures in the Model Piled Raft Test	85
3.6.1	<i>Preparation of foundation soil</i>	85
3.6.2	<i>Load test on model piled raft</i>	88
3.6.3	<i>Post test probing of the foundation soil</i>	91
3.7	Testing Programme	91
3.8	Index Properties of Kaolin Clay	93
3.9	Properties of Soil Specimens	93
3.10	Summary	96
Chapter 4	Experimental Results on Unpiled Raft and Single Piles	97
4.1	Introduction	97
4.2	Unpiled Raft	97
4.2.1	<i>Load-settlement behaviour of an unpiled raft in clay with $c_u = 40$ kPa (UR40)</i>	98
4.2.2	<i>Load-settlement behaviour of an unpiled raft in clay with $c_u = 50$ kPa (UR40)</i>	107
4.2.3	<i>Bearing capacity of the unpiled raft (P_u)</i>	112
4.2.4	<i>Interface contact pressure distribution</i>	116
4.3	Load Tests on Single Piles	123
4.4	Summary	130
Chapter 5	Load-Settlement Behaviour of Piled Rafts under Axial Load	132
5.1	Introduction	132

5.2	Test on Model 16-Piles Raft (16PB3L2)	132
5.3	Factors Affecting Settlement of Piled Raft	143
5.4.1	<i>Effect of number of piles on piled raft settlement</i>	143
5.4.2	<i>Effect of pile length</i>	139
5.4.3	<i>Effect of width of the piles</i>	148
5.4.4	<i>Effect of pile with similar pile capacity and stiffness but different pile dimensions</i>	152
5.4	Summary	154
Chapter 6	Load Distribution between Raft and Piles in Piled Rafts	155
6.1	Introduction	155
6.2	Effect of Number of Piles on the Distribution of Applied Load between Raft and Piles	156
6.3	Effect of Piles Length on Distribution of Applied Load between Raft and Piles	171
6.4	Effect of Pile with Similar Pile Capacity and Stiffness but Different Geometry on Distribution of Applied Load between Raft and Piles	176
6.5	Summary	180
Chapter 7	Development of Interface Contact Pressure and Piles Response in Piled Raft	182
7.1	Introduction	182
7.2	Mobilisation of Interface Contact Pressure with Raft Displacement	182
7.3	Load-Settlement Behaviour of Piles in the Piled Raft	189
7.4	Evaluating Effectiveness of Piles in Reducing Raft Settlement	194
7.5	Bearing Capacity of Piled Rafts (P_u^*)	199
7.6	Axial Force Variation along Pile Shaft	207
7.7	Summary	212
Chapter 8	Three Dimensional Finite Element Analyses of Unpiled Raft and Single Individual Piles	214
8.1	Introduction	214

8.2	Finite Element Modelling	215
8.3	Back Analysis of Unpiled Raft	228
8.4	Back Analysis of the Single Piles	234
8.5	Summary	242
Chapter 9	Three Dimensional Finite Element Analysis of Piled Rafts – and Comparison with Model Tests Results	243
9.1	Introduction	243
9.2	Finite Element Analyses of Model 9-Piles Raft (9PB3L2)	243
9.2.1	<i>Settlement response of model piled raft 9PB3L2</i>	245
9.2.2	<i>Distribution of applied load between raft and piles</i>	249
9.2.3	<i>Load transfer through raft bearing pressure</i>	251
9.2.4	<i>Load sharing between raft and piles</i>	254
9.3	Factors Affecting Settlement of Piled Raft	256
9.3.1	<i>Effect of number of piles</i>	256
9.3.2	<i>Effect of length of piles</i>	257
9.3.3	<i>Effect of pile dimensions for piles having similar capacity and stiffness</i>	259
9.4	Distribution of Applied Load between Raft and Piles	260
9.5	Contact Pressure versus Settlement Response	263
9.6	Load-Settlement Response of Individual Piles in Piled Rafts	274
9.7	Settlement Ratio (R_s)	280
9.8	Pile Load Contribution Factor (η)	282
9.9	Bearing Capacity of Piled Rafts (P_u^*)	286
9.10	Summary	288
Chapter 10	Conclusions and Recommendations for Future Work	276
10.1	Conclusions	290
10.2	Recommendations for Further Study	293
	References	295
Appendix A	Calibration Charts for Monitoring Instruments	307
Appendix B	Triaxial Test Data for Soil Specimens Retrieved After Model Tests	312

Appendix C	Interpretation of Bearing Capacity of Unpiled Raft from Model Test Data	315
Appendix D	Development of Interface Contact Pressure in Model Unpiled Raft Test	318
Appendix E	Differential Settlement of Piled Rafts from Model Tests	322
Appendix F	Mobilisation of Geotechnical Resistance of Piles in Model Tests	324
Appendix G	Development of Interface Contact Pressure in Model Piled Raft Tests	326
Appendix H	Interpretation of Bearing Capacity of Model Piled Rafts	330

LIST OF TABLES

Table	Description	Page
2.1	Specification of piles	18
3.1	Model piles dimensions and designations	69
3.2	Summary of experiments on piled raft specimens	93
3.3	Index properties of kaolin clay	93
3.4	Engineering properties of soil specimens	94
3.5	Properties of soil specimens retrieved after model tests	95
4.1	Deformation and stiffness of load (P) - central settlement (δ_c) response of an unpiled raft in clay with $c_u = 40$ kPa	101
4.2	Deformation and stiffness of load-settlement response at corner and quarter points of an unpiled raft in clay with $c_u = 40$ kPa	103
4.3	Deformation and stiffness of load - average settlement response of a unpiled raft in clay with $c_u = 40$ kPa	105
4.4	Deformation and stiffness of load-settlement response curves on soil with $c_u = 50$ kPa	108
4.5	Deduced bearing capacity of the unpiled raft by different methods	114
4.6	Estimated bearing capacity of the unpiled raft by different methods	116
4.7	Soil stiffness, k_s (kPa/mm) at corner and quarter points – PR40	122
4.8	Ultimate loads mobilised by different piles	124
4.9	Shaft and end bearing capacities of model piles	129
7.1	Contribution factor (η) at $\delta = 1.0$ mm; [$P_o = 4.3$ kN, available factor of safety against bearing capacity failure (FS) for unpiled raft $\cong 7$]	197

Table	Description	Page
7.2	Contribution factor (η) at $\delta = 2.0$ mm; [$P_o = 7.3$ kN, available factor of safety against bearing capacity failure (FS) for unpiled raft $\cong 4$]	198
7.3	Contribution factor (η) at $\delta = 3.0$ mm; [$P_o = 9.9$ kN, available factor of safety against bearing capacity failure (FS) for unpiled raft $\cong 3$]	198
7.4	Bearing capacity of piled rafts and back calculated α_G and α_{UR} values	205
7.5	The α_{UR} values from the present study and based on the equation proposed by de Sanctis & Madolini (2006)	206
7.6	Mobilised average unit skin friction (kPa) along pile shaft for B3L2 piles tested in isolation and in the 4- and 9-pile piled rafts at different pile head settlements	208
7.7	Mobilised average unit skin friction (kPa) along pile shaft for B2L4 piles tested in isolation and in the 4- and 9-pile piled rafts at different pile head settlements	212
8.1	Elastic properties adopted in the numerical analyses for model raft and model piles	220
8.2	Raft and soil properties	226
8.3	Summary of initial tangential stiffness and P_u for different cases of unpiled raft	230
8.4	Material properties adopted in the FE analyses for single isolated piles	236
8.5	Initial tangential stiffness (k_p) and Q_u for different individual piles from FE analyses and model tests using original soil parameters	238
8.6	Initial tangential stiffness (k_p) and Q_u for different individual piles from FE analyses and model tests using modified soil parameters	239
9.1	Material properties adopted in the numerical analyses	244
9.2	Settlement ratio (R_s) values from experimental and numerical study	283

Table	Description	Page
9.3	Contribution factor (η) at $\delta = 1.0$ mm; ($P_o = 4.10$ kN, available factor of safety against bearing capacity failure (FS) for unpiled raft $\cong 7.4$)	284
9.4	Contribution factor (η) at $\delta = 2.0$ mm; ($P_o = 8.21$ kN, available factor of safety against bearing capacity failure (FS) for unpiled raft $\cong 3.7$)	284
9.5	Contribution factor (η) at $\delta = 3.0$ mm; ($P_o = 12.31$ kN, available factor of safety against bearing capacity failure (FS) for unpiled raft $\cong 2.5$)	285
9.6	Bearing capacities of piled rafts and back calculated α_G and α_{UR} values	287

LIST OF FIGURES

Figure	Description	Page
2.1	Settlements for two adjacent buildings constructed using the two design approaches for the piled foundations(after Hansbo, 1993)	11
2.2	Central piles to reduce differential settlement (after Randolph, 1994)	12
2.3	Schematic design approach for differential settlement control (after Randolph, 1994)	13
2.4	Burland's simplified design concept (1995)	15
2.5	Piled raft foundation of MesseTurm Tower, Frankfurt, Germany (after Sommer <i>et al.</i> , 1991)	16
2.6	Settlements for Messeturm Tower, Frankfurt, Germany (after Tamaro, 1996)	16
2.7	Foundation plan and monitoring positions (after Yamashita <i>et al.</i> , 1994)	18
2.8	Measured settlements (after Yamashita <i>et al.</i> , 1994)	19
2.9	Foundation- Plan and cross section (after Katzenbach <i>et al.</i> , 1998)	20
2.10	Variation of loads and settlements with time (after Katzenbach <i>et al.</i> , 1998)	20
2.11	Plan of existing and new tanks (after de Sanctis & Russo, 2008)	22
2.12	Schematic view of eleven-storey office building in Aich prefecture, Japan (after Yamashita <i>et al.</i> , 2009)	24
2.13	Layout of piles and with location of monitoring devices at eleven-storey office building in Aich prefecture (after Yamashita <i>et al.</i> , 2009)	24
2.14	Schematic view of thirteen-storey office building in Osaka, Japan (after Yamashita <i>et al.</i> , 2009)	26
2.15	Layout of piles and monitoring devices at thirteen-storey office building in Osaka, Japan (after Yamashita <i>et al.</i> , 2009)	26
2.16	Schematic view of nineteen-storey office building in Kogoshima, Japan (after Yamashita <i>et al.</i> , 2009)	28

Figure	Description	Page
2.17	Layout of piles and grid-form soil-cement walls with location of monitoring devices at nineteen-storey office building in Kogoshima, Japan (after Yamashita <i>et al.</i> , 2009)	28
2.18	Amuplaza building in Kagoshima city, Kyushu, Japan (after Sonada <i>et al.</i> , 2009)	30
2.19	Load settlement behaviour referred to prototype scale (after Fioravante, 1998)	33
2.20	Simplified load-settlement curve for preliminary analysis (after Poulos, 2001)	38
2.21	Representation of piled strip problem (after Poulos, 1991)	42
2.22	Basic features of the model for piled raft (after Russo, 1998)	46
2.23	Typical piled raft and its numerical modelling (after Anagnostopoulos & Geogiadis, 1998)	47
2.24	FE model of a piled raft (Kim <i>et al.</i> , 2001)	47
2.25	Analytical model for raft-soil pile system by Yamashita <i>et al.</i> (after Yamashita <i>et al.</i> , 1998)	48
2.26	Numerical representation of piled raft (after Clancy and Randolph, 1993)	50
2.27	Plate-beam-spring modeling of piled raft(after Kitiyodom & Matsumoto, 2002)	51
2.28	Analytical model of typical piled raft (after Brown & Wiesner, 1975)	54
2.29	Arrangement of soil elements and pile elements for compatibility with the raft (after Hain & Lee, 1978)	56
2.30	Force considered to act on the piles and on the soil (after Ta & Small, 1996)	58
3.1	Model raft	68
3.2	Schematics of model pile	71
3.3	Schematics of consolidometer and instrumentation	73
3.4	Schematic diagram showing the set-up for applying central point load	75

Figure	Description	Page
3.5	Monitoring positions of displacement transducers on the raft	78
3.6	Monitoring positions of stain gages with in a pile	80
3.7	Locations of miniature pressure transducers beneath the raft	82
3.8	Pile Installation Guide used to install piles vertically for 4 piles raft	84
3.9	Schematic diagram of piles installation process	89
3.10	Locations for sample collection and in-situ probing	95
4.1	Applied central point load (P) versus central settlement (δ_c) of an unpiled raft in clay with $c_u = 40$ kPa	99
4.2	Applied central point load (P) versus central settlement (δ_c) of an unpiled raft in clay with $c_u = 40$ kPa – in different load cycles	100
4.3	Applied central point load (P) versus settlement at corner points of an unpiled raft in clay with $c_u = 40$ kPa	102
4.4	Applied central point load (P) versus settlement at quarter points of an unpiled raft in clay with $c_u = 40$ kPa	103
4.5	Applied central point load (P) versus average settlement (δ_{avg}) of an unpiled raft in clay with $c_u = 40$ kPa	104
4.6	Applied central point load (P) versus average settlement (δ_{avg}) of an unpiled raft in clay with $c_u = 40$ kPa - in different load cycles	105
4.7	Applied central point load (P) versus differential settlement ($\Delta\delta$) of an unpiled raft in clay with $c_u = 40$ kPa	106
4.8	Applied central point load (P) versus differential settlement ($\Delta\delta$) of an unpiled raft in clay with $c_u = 40$ kPa - in different load cycles	107
4.9	Applied central point load (P) versus central settlement (δ_c) of an unpiled raft in clay with $c_u = 50$ kPa	109
4.10	Applied central point load (P) versus settlement (δ) at corners of an unpiled raft in clay with $c_u = 50$ kPa	109
4.11	Applied central point load (P) versus settlement (δ) at quarter points of an unpiled raft in clay with $c_u = 50$ kPa	110

Figure	Description	Page
4.12	Applied central point load (P) versus average settlement (δ_{avg}) of an unpiled raft in clay with $c_u = 50$ kPa	110
4.13	Applied central point load (P) versus differential settlement ($\Delta\delta$) of an unpiled raft in clay with $c_u = 50$ kPa	112
4.14	Applied central point load (P) versus average settlement δ_{avg} of an unpiled raft in clay with $c_u = 40$ kPa from model test and representative single stage response	113
4.15	Applied central point load (P) versus average settlement δ_{avg} of an unpiled raft in clay with $c_u = 50$ kPa from model test and representative single stage response	113
4.16	Applied central point load (P) versus interface contact pressure (σ_c) at corner points in first load cycle	117
4.17	Applied central point load (P) versus interface contact pressure (σ_c) at quarter points in first load cycle	118
4.18	Applied central point load (P) versus average interface contact pressure at corner and quarter points in first load cycle	119
4.19	Measured interface contact pressure (σ_c) versus raft settlement (δ) response in first load cycle	120
4.20	Variation of soil spring stiffness along diagonal of the raft	121
4.21	Applied central point load (P) versus average settlement (δ_{avg}) for UR40 and UR50	123
4.22	Pile head load versus pile head settlement for single isolated pile B3L4 ($B = 30$ mm & $L = 400$ mm)	125
4.23	Pile head load versus pile head settlement for piles B2L2, B3L2 and B2L4	125
4.24	Variation of end bearing and shaft resistance with pile head settlement for pile B2L2 ($B = 20$ mm, $L = 200$ mm)	127
4.25	Variation of end bearing and shaft resistance with pile head settlement for pile B2L4 ($B = 20$ mm, $L = 400$ mm)	127
4.26	Variation of end bearing and shaft friction resistance with pile head settlement for pile B3L2 ($B = 30$ mm, $L = 200$ mm)	128

Figure	Description	Page
4.27	Variation of end bearing and shaft friction resistance with pile head settlement for pile B3L4 ($B = 30$ mm, $L = 400$ mm)	128
5.1	Applied central point load (P) versus central settlement (δ_c) response of 16-piles raft (16PB3L2)	133
5.2	Applied central point load (P) versus settlement (δ) at 3 corners of 16-piles raft (16PB3L2)	134
5.3	Applied central point load (P) versus differential settlement ($\Delta\delta$) of 16-piles raft (16PB3L2)	135
5.4	Mobilisation of geotechnical resistance of piles with applied load on raft in 16PB3L2	137
5.5	Mobilisation of interface contact pressure with applied load on raft at corners in 16PB3L2	138
5.6	Mobilisation of interface contact pressure with applied load on raft at quarter points in 16PB3L2	139
5.7	Reduction in interface contact pressure in 16PB3L2 compared to unpiled raft	140
5.8	Variation of load sharing between piles and raft with applied load in 16PB3L2	142
5.9	Applied central point load (P) versus central settlement (δ_c) of 4-, 9- and 16-piles rafts with piles of $B = 30$ mm and $L = 200$ mm	144
5.10	Applied central point load (P) versus central settlement (δ_c) of 4- and 9-piles rafts with piles of $B = 20$ mm, $L = 200$ mm & 400 mm	146
5.11	Applied central point load (P) versus central settlement (δ_c) of 4- and 9-piles rafts with piles of $B = 30$ mm and $L = 400$ mm	147
5.12	Applied central point load (P) versus central settlement (δ_c) of 4- and 9-piles rafts with piles of the same B of 30 mm, & $L = 200$ mm and $L = 400$ mm	149
5.13	Applied central point load (P) versus central settlement (δ_c) of 4- and 9-piles rafts with piles of the same L of 200 mm and $B = 20$ mm & $B = 30$ mm	150
5.14	Load (P) versus central settlement (δ_c) of 4- and 9-piles rafts with piles of the same L of 400 mm and $B = 20$ mm & $B = 30$ mm	152

Figure	Description	Page
5.15	Applied central point load (P) versus central settlement (δ_c) of 4- and 9-piles rafts with piles of the same capacity but different geometry	153
6.1	Layout of piles and instrumented piles in 4-, 9- and 16-piles rafts	156
6.2	Variation of mobilized geotechnical resistance of individual piles of $B = 30$ mm and $L = 200$ mm in 4-, 9- and 16-piles rafts	157
6.3	Variation of mobilised total resistance by all piles with applied load on raft in 4-, 9- and 16-piles rafts with piles of $B = 30$ mm and $L = 200$ mm	159
6.4	Mobilisation of interface contact pressure with applied load at quarter points in 4-, 9- and 16-piles rafts with $B = 30$ mm and $L = 200$ mm	159
6.5	Mobilisation of interface contact pressure with applied load at corners in 4-, 9- and 16-piles rafts with $B = 30$ mm and $L = 200$ mm	161
6.6	Variation of raft bearing resistance with applied load in 4-, 9- and 16-piles rafts with $B = 30$ mm and $L = 200$ mm	163
6.7	Variation of percentage load carried by piles and raft with the applied load (P) in 4-, 9- and 16-piles raft with piles of $B = 30$ mm and $L = 200$ mm	163
6.8	Applied central point load (P) versus mobilized geotechnical resistance of piles of $B = 30$ mm and $L = 400$ mm in 4- and 9-piles rafts	166
6.9	Applied central point load (P) versus mobilised geotechnical resistance by all piles in 4- and 9-piles rafts with piles of $B = 30$ mm and $L = 400$ mm	167
6.10	Mobilisation of interface contact pressure (σ_c) with applied load (P) at quarter points in 4- and 9- piles rafts with $B = 30$ mm and $L = 400$ mm	168
6.11	Mobilisation of interface contact pressure (σ_c) with applied load (P) at corners in 4- and 9- piles rafts with $B = 30$ mm and $L = 400$ mm	169
6.12	Variation of raft bearing resistance with applied load in 4- and 9-piles rafts with $B = 30$ mm and $L = 400$ mm	170
6.13	Percentage load sharing between piles and raft in 4- and 9-piles rafts with $B = 30$ mm and $L = 400$ mm	171

Figure	Description	Page
6.14	Applied central point load (P) versus mobilized geotechnical resistance of piles of $B = 20$ mm and $L = 200$ mm & 400 mm in 4- and 9-piles rafts	173
6.15	Applied central point load (P) versus mobilised geotechnical resistance by all piles in 4- and 9-piles rafts with piles of $B = 20$ mm and $L = 200$ & 400 mm	173
6.16	Variation of raft bearing resistance with applied load in 4- and 9-piles rafts with piles of $B = 20$ mm and $L = 200$ & 400 mm	175
6.17	Percentage load sharing between piles and raft in 4- and 9-piles rafts with piles of $B = 20$ mm and $L = 200$ mm and & 400 mm	176
6.18	Load (P) versus mobilized geotechnical resistance of piles of $B=30$ mm & $L=200$ mm and $B=20$ mm & $L=400$ mm in 4- and 9-piles rafts	177
6.19	Variation of raft bearing resistance with applied load in 4- and 9-piles rafts with piles of $B=30$ mm & $L=200$ mm and $B=20$ mm & $L=400$ mm	178
6.20	Percentage load sharing between piles and raft in 4- and 9-piles rafts with piles of $B=30$ mm & $L =200$ mm and $B=20$ mm & $L=400$ mm	179
7.1	Mobilisation of interface contact pressure (σ_c) with settlement (δ) at quarter points in first load cycle in all 4-piles rafts	183
7.2	Mobilisation of interface contact pressure (σ_c) with settlement (δ) at quarter points in first load cycle in all 9-piles rafts	184
7.3	Mobilisation of interface contact pressure (σ_c) with settlement (δ) at quarter points in first load cycle in 4-, 9- and 16-piles raft with piles of $B = 30$ mm and $L = 200$ mm	184
7.4	Mobilisation of interface contact pressure (σ_c) with settlement (δ) at corners in first load cycle in all 4-piles rafts	185
7.5	Mobilisation of interface contact pressure (σ_c) with settlement (δ) at corners in first load cycle in all 9-piles rafts	186

Figure	Description	Page
7.6	Mobilisation of interface contact pressure (σ_c) with settlement (δ) at quarter points and corners in first load cycle in the unpiled raft, 9PB3L2 and 16PB3L2	188
7.7	Pile head load versus pile head displacement response of pile B3L2 ($B = 30$ mm & $L = 200$ mm) in single individual case, 4- and 9-piles rafts	189
7.8	Pile head load versus pile head displacement response of pile B2L3 ($B = 30$ mm & $L = 200$ mm) in single individual case and 16-piles rafts	190
7.9	Pile head load versus pile head displacement response of pile B2L2 ($B = 20$ mm & $L = 200$ mm) in single individual case, 4- and 9-piles rafts	191
7.10	Pile head load versus pile head displacement response of pile B2L4 ($B = 20$ mm & $L = 400$ mm) in single individual case, 4- and 9-piles rafts	192
7.11	Pile head load versus pile head displacement response of pile B2L4 ($B = 30$ mm & $L = 400$ mm) in single individual case, 4- and 9-piles rafts	193
7.12	Schematics showing the determination of pile contribution factor η	195
7.13	Variation of contribution factor (η) of piles with spacing	196
7.14	The applied load (P) versus average settlement (δ_{avg}) response of 16PB3L2 in three load cycles and the representative single stage response	200
7.15	The applied load (P) versus average settlement (δ_{avg}) response of 16PB3L2 and unpiled raft in three load cycles	201
7.16	Variation of axial force along the pile shaft at a pile head settlement of 0.5 mm for pile B3L2 ($B = 30$ mm & $L = 200$ mm) in single individual case, 4- and 9-pile piled rafts	207
7.17	Variation of axial force along the pile shaft at a pile head settlement of 2.0 mm for pile B3L2 ($B = 30$ mm & $L = 200$ mm) in single individual case, 4- and 9-pile piled rafts	209
7.18	Variation of axial force along the pile shaft at a pile head settlement of 0.5 mm for pile B2L4 ($B = 20$ mm & $L = 400$ mm) in single individual case, 4- and 9-pile piled rafts	210

Figure	Description	Page
7.19	Variation of axial force along the pile shaft at a pile head settlement of 2.0 mm for pile B2L4 ($B = 20$ mm & $L = 400$ mm) in single individual case, 4- and 9-pile piled rafts	211
8.1	Linear Drucker-Prager model (after Drucker and Prager, 1952)	217
8.2	Schematics of the test set-up for determining equivalent stiffness of the raft	220
8.3	"Hard" contact model pressure-over closure relationship	221
8.4	Finite element mesh pattern and boundary conditions for unpiled raft	223
8.5	Applied central point load (P) versus average settlement response (δ_{avg}) using Mohr-Coulomb and Drucker-Prager models	225
8.6	Difference in the back calculated load using contact pressure and applied load	226
8.7	Interface contact pressure (σ_c) and settlement of a flexible strip footing	227
8.8	Interface contact pressure (σ_c) and settlement (δ) of a rigid strip footing	228
8.9	Applied central point load (P) versus predicted central settlement (δ_{avg}) adopting different values of Young's modulus for soil	229
8.10	Applied central point load (P) versus predicted differential settlement ($\Delta\delta$) adopting different values of Young's modulus for soil	230
8.11	Interface contact pressure (σ_c) versus settlement (δ) response at corners obtained from FE analyses and model tests	232
8.12	Interface contact pressure (σ_c) versus settlement (δ) response at quarter points obtained from FE analyses and model tests	232
8.13	Mobilisation of interface contact pressure (σ_c) with applied load (P) at corners for unpiled raft obtained from FE analyses and model test	233
8.14	Mobilisation of interface contact pressure (σ_c) with applied load (P) at corners for unpiled raft obtained from FE analyses and model test	234

Figure	Description	Page
8.15	Finite element mesh pattern for single piles	235
8.16	The computed and measured pile head load versus pile head displacement response of isolated single pile B2L2 ($B=20$ mm & $L=200$ mm)	236
8.17	The computed and measured pile head load versus pile head displacement response of isolated single pile B2L4 ($B=30$ mm & $L=200$ mm)	237
8.18	The computed and measured pile head load versus pile head displacement response of isolated single pile B2L4 ($B=20$ mm & $L=400$ mm)	237
8.19	The computed and measured pile head load versus pile head displacement response of isolated single pile B2L4 ($B=30$ mm & $L=400$ mm)	239
8.20	The computed and measured shaft and end bearing resistance versus pile head displacement response of isolated single pile B2L2 ($B=20$ mm & $L=200$ mm)	240
8.21	The computed and measured shaft and end bearing resistance versus pile head displacement response of isolated single pile B2L2 ($B=30$ mm & $L=200$ mm)	240
8.22	The computed and measured shaft and end bearing resistance versus pile head displacement response of isolated single pile B2L2 ($B=20$ mm & $L=400$ mm)	241
8.23	The computed and measured shaft and end bearing resistance versus pile head displacement response of isolated single pile B2L2 ($B=30$ mm & $L=400$ mm)	241
9.1	Finite element mesh pattern for piled raft specimens for 9-piles raft with piles of $B = 30$ mm & $L = 200$ mm	245
9.2	Applied central point load (P) versus central settlement (δ_c) response of 9PB3L2 obtained from FE analyses and model test	246
9.3	Applied central point load (P) versus average settlement (δ_{avg}) response of 9PB3L2 obtained from FE analyses and model test	247
9.4	Applied central point load (P) versus differential settlement ($\Delta\delta$) response of 9PB3L2 obtained from FE analyses and model test	249
9.5	Variation of mobilised geotechnical resistance of individual piles of $B = 30$ mm and $L = 200$ mm 9-piles raft obtained from FE analyses and model test	250

Figure	Description	Page
9.6	Mobilisation of interface contact pressure (σ_c) with applied load (P) at quarter points 9-piles raft with $B = 30$ mm and $L = 200$ mm obtained from FE analyses and model test	252
9.7	Mobilisation of interface contact pressure (σ_c) with applied load (P) at corner points 9-piles raft with $B = 30$ mm and $L = 200$ mm obtained from FE analyses and model test	253
9.8	Variation of raft bearing resistance with applied load in 9-piles raft with $B = 30$ mm and $L = 200$ mm obtained from FE analyses and model test	254
9.9	Variation of percentage load carried by piles and raft with the applied load (P) in 9-piles raft with piles of $B = 30$ mm and $L = 200$ mm obtained from FE analyses and model test	255
9.10	Applied central point load (P) versus central settlement (δ_c) of 4-, 9- and 16-piles rafts with piles of $B = 30$ mm and $L = 200$ mm obtained from FE analyses	257
9.11	Load (P) versus central settlement (δ_c) of 4- and 9-piles rafts with piles of $B=20$ mm, $L=200$ mm & 400 mm obtained from FE analyses	258
9.12	Load (P) versus central settlement (δ_c) of 4- and 9-piles rafts with piles of the same capacity but different geometry	259
9.13	Variation of mobilized geotechnical resistance of individual piles of $B = 30$ mm and $L = 200$ mm in 4-, 9- and 16-piles rafts obtained from FE analyses	260
9.14	Variation of raft bearing resistance with applied load in 4-, 9- and 16-piles rafts with $B = 30$ mm and $L = 200$ mm obtained from FE analyses and model tests	262
9.15	Variation of percentage load carried by piles and raft with the applied load (P) in 4-, 9- and 16-piles raft with piles of $B = 30$ mm and $L = 200$ mm obtained from FE analyses	263
9.16	Interface contact pressure (σ_c) versus settlement (δ) response at quarter points for all 4-piles rafts obtained from FE analyses and model tests	264
9.17	Interface contact pressure (σ_c) versus settlement (δ) response at quarter points for all 9-piles rafts obtained from FE analyses and model tests	266

Figure	Description	Page
9.18	Interface contact pressure (σ_c) versus settlement (δ) response at quarter points for all 4-, 9- and 16-piles rafts with piles of $B=30\text{mm}$ & $L=200\text{ mm}$ obtained from FE analyses and model tests	268
9.19	Interface contact pressure (σ_c) versus settlement (δ) response at quarter points for all 4-piles rafts obtained from FE analyses and model tests	270
9.20	Interface contact pressure (σ_c) versus settlement (δ) response at quarter points for all 9-piles rafts obtained from FE analyses and model tests	271
9.21	Interface contact pressure (σ_c) with settlement (δ) at quarter points and corners in the unpiled raft, 9PB3L2 and 16PB3L2 obtained from FE analyses and model tests	273
9.22	Pile head load versus pile head displacement response of pile B2L2 ($B = 20\text{ mm}$ & $L = 200\text{ mm}$) single isolated case, in 4- and 9-piles rafts obtained from FE analyses and model tests	275
9.23	Pile head load versus pile head displacement response of pile B3L2 ($B = 30\text{ mm}$ & $L = 200\text{ mm}$) single isolated case, in 4- and 9-piles rafts obtained from FE analyses and model tests	276
9.24	Pile head load versus pile head displacement response of pile B3L2 ($B = 30\text{ mm}$ & $L = 200\text{ mm}$) single isolated case, in 16-piles rafts obtained from FE analyses and model tests	277
9.25	Pile head load versus pile head displacement response of pile B2L4 ($B = 20\text{ mm}$ & $L = 400\text{ mm}$) single isolated case, in 16-piles rafts obtained from FE analyses and model tests	278
9.26	Pile head load versus pile head displacement response of pile B3L4 ($B = 30\text{ mm}$ & $L = 400\text{ mm}$) single isolated case, in 16-piles rafts obtained from FE analyses and model tests	279
9.27	Variation of settlement ratio (R_s) with s/d ratio	281
9.28	Variation of contribution factor (η) of piles with s/B ratio	286

LIST OF PHOTOGRAPHS

Photo	Description	Page
3.1	Model raft	67
3.2	Different size piles used in this study	70
3.3	Set-up of the loading system for applying central point load on raft	76
3.4	Test gauge to monitor consolidation pressure	78
3.5	Monitoring positions of displacement transducers on the raft	79
3.6	Locations of miniature pressure transducers on the underside of the raft	81
3.7	Pile installation process	90

LIST OF SYMBOLS

A_b	=	area of the pile base
A_G	=	area enclosed by peripheral piles
A_p	=	pile cross-sectional area
A_R	=	area of the raft
A_s	=	surface area of the shaft
B	=	width of pile
B_R	=	width of raft
c_c	=	compression index
c_r	=	recompression index
c_u	=	undrained shear strength
d	=	diameter of the piles
D	=	intercept of the yield surface
E	=	Young's modulus
E_p	=	Young's modulus of elasticity for the model piles
E_r	=	equivalent Young's modulus of elasticity of the raft
E_s	=	Young's modulus of soil
E_{sav}	=	average soil Young's modulus along pile shaft
E_{sb}	=	soil Young's modulus of bearing stratum below pile tip
E_{sl}	=	soil Young's modulus at level of pile tip
E_u	=	initial tangent modulus
e_o	=	initial void ratio
FS	=	factor of safety
f_s	=	unit skin friction
G_s	=	specific gravity
K	=	ratio of the yield stress in triaxial tension to the yield stress in triaxial compression
k^*	=	ratio of applied pressure to the average raft settlement
k_o	=	gradient at zero load of P - δ curve
k_p	=	average slope of pre-failure load-displacement curve
k_s	=	secant soil stiffness

k_s	=	permeability
L	=	length of pile
L_r	=	in-plane raft length
n	=	number of piles
N_c	=	bearing capacity factor
P	=	applied central point load on raft
P_0	=	design load in excess of P_l
P_l	=	load carried by unpiled raft at target settlement
p_a	=	average applied pressure on raft
p_a	=	average applied pressure
P_d	=	design load
P_r	=	load carried by the raft
P_t	=	total applied load
P_u	=	bearing capacity of unpiled raft
P_u^*	=	bearing capacity of piled raft
Q	=	column load
Q_l	=	axial load at pile head
Q_4	=	axial load at 20 mm above pile tip
Q_b	=	end bearing capacity
Q_d	=	design load
Q_g	=	load carried by piles in the piled raft
Q_R	=	load carried by the raft corresponding to allowable settlement
Q_r	=	reduced load calculated from the actual column load
Q_s	=	shaft resistance
Q_u	=	bearing capacity of single individual pile
Q_u^*	=	bearing capacity of pile group
q_b	=	end bearing resistance
R_s	=	settlement ratio
r_c	=	average radius of pile cap
r_o	=	radius of pile
s	=	spacing between the piles
s_c	=	shape factor
ν	=	poisson's ratio
α	=	adhesion factor

δ	=	settlement
η	=	pile load contribution factor
β	=	slope of the linear yield surface in the $P-t$ stress plane
ξ	=	group efficiency factor
$\Delta\delta$	=	differential settlement
δ_a	=	allowable settlement
δ_{avg}	=	average settlement
δ_c	=	central settlement
σ_c	=	interface contact pressure
α_G	=	modifying coefficients for the pile group when combined in a piled raft
β_p	=	pile –soil interaction factor
β_s	=	soil-pile interaction factor
α_{UR}	=	modifying coefficients for the failure load of the raft when combined in a piled raft
σ_{vo}'	=	effective overburden pressure
δ_a	=	allowable settlement
δ_d	=	total Settlement
ϕ_u	=	undrained friction angle

SUMMARY

Shallow foundation such as rafts is often the most economical and viable solution when the soil near the ground surface is competent and provides sufficient bearing capacity. If the raft settlement is excessive, a limited number of piles can be introduced to reduce the raft settlements. The role of piles in raft settlement reduction is not clearly understood. In this study, model tests were conducted to unravel the effects of various piled raft parameters in reducing raft settlements.

The experimental results revealed that the number of piles added was the primary factor affecting raft settlement. For a given pile number, stiffer piles would cause the largest reduction in settlement. The effectiveness of the piles was also influenced by pile spacing. Piles of different length and width but the same capacity and stiffness have similar effect on raft settlement; particularly when the applied load was well below the raft bearing capacity. The model tests also revealed that although short piles could help to reduce raft settlement at low applied load level, they were ineffective in enhancing the raft bearing capacity.

The larger pile stiffness caused the piles to carry a large proportion of the applied load, thus reducing the load transmitted through raft bearing pressure, initially. Thus, significant increase in bearing pressure, and hence larger raft settlement, would occur only when pile capacities had been substantially exhausted. The results from the model test indicated that when piles were well spaced, at least 75% of the single pile capacity could be mobilised in the piled raft under typical working load condition; and could reach 100% in many instances.

The experimental data showed that the interface contact pressure-displacement relationship or soil spring curve at raft-soil interface was highly non-linear. A unique soil spring relationship appeared to apply to all nine rafts at moderate stress level.

3-D finite element analyses of the model piled rafts yielded load-settlement curves which were in good agreement with the test data. However, significant differences existed between the test results and the finite element output in some key aspects such as the raft-soil interface response and load distribution between raft and piles. Further study is needed to uncover the reasons for these observed discrepancies.

Chapter 1

Introduction

1.1 Background

Shallow foundation system such as rafts could be viable when the foundation soil near the ground surface is competent (e.g. stiff clay or dense sand). However, even when the factor of safety against bearing capacity failure of the raft is adequate, the raft may settle excessively. Average settlements of the raft can be controlled by reducing the bearing pressures in relation to the bearing capacity of the soil. This can be achieved by placing the raft at the bottom of an excavation, or by introducing piles. In addition, introducing piles would also help to reduce differential settlements as well. Differential settlements can also be reduced by thickening the raft. Among the different methods for reducing average and differential settlements of the raft, the inclusion of piles could potentially be a most effective and economic method for certain favourable conditions.

In the past few years, there has been an increasing trend towards the use of piles as settlement reducers. In foundations where both the raft and piles are used, the performance of the raft is improved with only a limited number of piles to reduce the settlements. When piles are used to reduce settlements, the number of piles and their locations are selected in such a way as to bring the settlements within acceptable limits. The piles in this case do not act primarily as load carrying members, but act as settlement reducers. The number of piles required in this case is significantly less than the number of piles required to carry the entire applied load, provided that the load-carrying capacity of each pile is fully mobilized. Thus the foundation cost can considerably be reduced.

The concept of using piles as settlement reducers is first proposed by Burland *et al.* (1977). Subsequently, a number of comprehensive reports have been published on

the use of piles as settlement reducers (Burland and Karla, 1986; Randolph, 1994; Russo and Viggiani, 1998; Viggiani, 2001; Poulos, 2001; de Sanctis *et al.*, 2002; Mandolini *et al.*, 2005). Also, a number of case histories have been published on the use of piles as settlement reducers which reveal that the number of piles required to reduce raft settlements to within acceptable limits are considerably less than the number required to carry the entire applied load (Hansbo and Källström, 1983; Katzenbach *et al.*, 2000; Love 2003; Reul and Randolph, 2003; Russo *et al.*, 2004; de Sanctis and Russo 2008). Nevertheless the traditional capacity based design approach is still dominant among practising engineers owing to the lack of knowledge about the complicated interactions between the raft, soil and piles.

The available design philosophies for piled rafts have been clearly defined by Randolph (1994). These include the conventional approach, creep piling and differential settlement control methods. In the conventional design approach, the foundation is designed essentially as a pile group. The piles are regularly placed over the entire foundation area. Some allowance is made for the load transmitted directly from the raft to the soil below it. Raft is expected to contribute towards the ultimate capacity of the foundation. The advantage of this approach is the reduction in numbers of piles required, as some part of the structural load is assumed to be directly transmitted to the soil from the raft. The creep piling approach is originally proposed for relatively soft cohesive soils by Hansbo & Källström (1983). In this approach each pile is designed to operate at a working load of 70 to 80 % of the ultimate load. The foundation is designed as a raft foundation and sufficient number of piles is included to reduce the net contact pressure below the preconsolidation pressure. In both of the above approaches, the piles are distributed under the raft with the main aim of reducing absolute settlements. On the other hand, in the differential settlement control approach due consideration is given to differential settlements. In this approach the piles are located strategically in order to reduce the differential settlements rather than to reduce overall settlements.

In addition to the above mentioned design approaches, there is one more extreme version of creep piling approach (Poulos, 2000; Poulos, 2001). In this approach, some or all of the piles are assumed to operate at their ultimate load capacity. To adopt this extreme creep piling approach or to economise using piles as settlement

reducers, the behaviour of piles in a piled raft at working loads need to be known with sufficient confidence.

Based on case histories (Cooke *et. al.*, 1981; Yamashita and Kakurai, 1991; Yamashita *et. al.*, 1994; Katzenbach *et. al.*, 1998; Reul and Randolph, 2003; Russo *et. al.*, 2004; de Sanctis and Russo, 2008), experimental studies (Cooke, 1986; Horikoshi, 1995; Fioravante, 1998; Cao, 1998) and numerical studies available in the literature, it is clear that the behaviour of piles in piled raft is considerably different from the behaviour of individual piles. In order to economically design piles as settlement reducers, the understanding of pile behaviour at working loads of a piled raft foundation is essential. Detailed experimental investigations providing insights of pile behaviour at working loads of piled raft foundation are rare.

The design of piled raft foundation involves several key variables such as load combinations, raft thickness, location, number, and size of piles. To optimise these parameters, many iterations are required before the final phase of design. An exact solution of this complex problem involving several variables is extremely difficult. To do a detailed 3D analysis for each iteration requires much time and resources. Hence, it is common to adopt approximate computer based methods to assess piling requirements. In some of these approximate computer based methods soil and piles are represented using springs. The spring constants are determined using theoretical or empirical solutions. Experimental investigations to study the behaviour of this soil and pile springs are rare.

1.2 Objective of Present Research

As discussed in the previous section, sufficient information is not available on the behaviour of piles at working loads of the piled raft. Studies of how settlements are being modified due to the introduction of piles of different geometry and group configuration will provide insight into the use of piles primarily to reduce settlements. Understanding the behaviour of piles at working loads of the piled raft will help in developing an economic and efficient design using piles as settlement reducers. This will ultimately lead to economy in foundation cost. Therefore, the main objective of this research is to investigate the load-settlement behaviour, and

load-transfer mechanism of the raft having sufficient bearing capacity and supported on piles.

1.3 Scope of Present Work

The present work consists of both experimental investigation and 3-D finite element analyses of piled rafts. The experimental investigation was carried out in clayey soil prepared in laboratory. Dry kaolin powder was used to prepare the model ground. The ground was comprehensively characterised by standard test methods. Tests were carried out for unpiled raft, single isolated piles and piled raft specimens. The study was carried out using relatively rigid rafts having raft-soil stiffness ratio approximately equal to 1.85. The effect of pile length and pile size was investigated using four different combinations of pile size and length. The slenderness ratio of the model piles was between 6.67 and 20 and pile-soil stiffness ratio (E_p/E_s) for model piles was about 8600, which was within the practical range of 100 to 10000 (Randolph, 1994). The key data obtained from the laboratory investigation included the load settlement of the raft, raft-soil interface contact pressure and axial load carried by piles.

Numerical simulations of the model test were carried out using the 3-D finite element program Abaqus. The input parameters for the numerical models were calibrated using the data obtained from the unpiled raft and single isolated piles load test data. The calibrated soil parameters were then applied to study the load-transfer mechanism of piled rafts.

1.4 Outline of Thesis

Chapter 1 provides a brief introduction, laying out the need for the present study and objective of the study.

In Chapter 2, a literature review of related issues and motivation for the current study are presented. The review covers the concept of piled rafts, different design philosophies for the design of the piled rafts, laboratory and in-situ studies on the piled rafts and analysis of the rafts & piled rafts.

Chapter 3 describes the experimental set-up, instrumentation and the test procedures. This chapter also lays out the testing programme.

Chapter 4 presents the experimental results on unpiled rafts and single piles.

Chapter 5 presents the experimental load settlement behaviour of piled rafts.

Chapter 6 covers the distribution of applied load between raft and piles in piled rafts obtained from model tests.

Chapter 7 presents the development of interface contact pressure and response of piles in piled raft.

Chapter 8 provides an overview of the three dimensional finite element analysis of unpiled rafts and single individual piles and the calibration of numerical model.

Chapter 9 presents the three dimensional finite element analysis of piled rafts and comparison of model test results with results obtained from FE analyses.

Chapter 10 summarizes the main conclusions from the present study and makes recommendations for future works.

Chapter 2

Literature Review

2.1 Introduction

In situations where raft foundation alone can provide sufficient bearing capacity and settles excessively, introducing piles could be most efficient and economical approach to control the settlements. To design piles economically for reducing settlements, it is necessary to know the distribution of applied load between raft and piles. Considerable research work has been carried out on piled rafts foundations. In this chapter, a brief review of the concept of piled rafts, piled raft design philosophies, in-situ, experimental, and numerical studies is presented.

2.2 Concept of Piled Rafts

In a free standing pile group where the cap is not in contact with the soil, the piles are assumed to carry the entire applied loads. In a piled raft foundation, both the raft and piles carry the applied loads. As discussed in previous section, when the raft foundation experiences excessive settlements, piles can be introduced to reduce the settlements. Utilising the load carrying capacity of the foundation, the pile geometry and group configuration should be selected in such way that the resulting raft settlements are within acceptable limits. The strategically placed piles also reduce the bending moments and shear forces within the raft compared to unpiled raft. This will generally produce more economical foundation.

2.2.1 Favourable and unfavourable site conditions for piled rafts

The most favourable condition for using piled rafts or piles to reduce settlements is when the raft itself has sufficient factor of safety against the bearing capacity failure, but the average settlement and/or differential settlements of the raft exceed

the permissible values. Poulos (1991) has examined a number of idealised soil profiles, and found that the following situations may be favourable.

1. Soil profiles consisting of relatively stiff clays
2. Soil profiles consisting of relatively dense sands

In both situations usually raft alone can provide a major proportion, if not all, of the required bearing capacity. The piles added will improve the performance of the foundation system, rather than providing a major means of support.

Conversely, there are situations that are unfavourable for the piled raft foundations. These typically occur when weak soils such as soft clay or loose sand are found near the surface. Similar unfavourable conditions may also include sub soils that contain soft compressible layers at shallow depths. Soils that are undergoing consolidation settlements or are susceptible to swelling are also not favourable to the adaptation of piled raft foundation.

In soft clay and loose sand, the raft may not be able to provide significant load carrying capacity and stiffness. The long-term settlement of the compressible layers may reduce the contribution of the raft to the long-term stiffness of the foundation. Consolidation settlements such as those due to dewatering or shrinkage of an active clay soil may result in a loss of contact between the raft and the soil, thus increasing the load on the piles, and leading to increased settlement of the foundation system. Cooke *et al.* (1981) reported that due to consolidation the proportion of load carried by piles could increase from 55% to 75% of the total load and the proportion of load carried by raft was decreased. In the case of swelling soil, tensile forces may be induced in the piles because of the action of the swelling soil on the raft.

2.2.2 Different categories of piled raft foundations

Russo and Viggiani (1998) and Viggiani (2000) have identified two categories of piled raft foundations. In the first category, piles are required to increase the overall factor of safety against bearing capacity failure and in the second category piles are required only to reduce the settlements.

Small piled rafts: In this category of piled rafts, the primary reason for adding piles is to increase the overall factor of safety against bearing capacity failure. It typically consists of rafts having widths between 5 m and 15 m. In this case, the width of the raft is less than the length of the piles.

Large piled rafts: In this second category of piled rafts, the raft itself has sufficient bearing capacity to carry the applied load with a reasonable factor of safety, but piles are required to reduce the average settlement or differential settlement. In such cases the width of the raft is large compared to the length of piles.

2.3 Piled Raft Design Philosophies

The following are some of the different design philosophies with respect to the piled rafts discussed by Randolph (1994).

2.3.1 Conventional approach

In the conventional approach, the piles are distributed uniformly over the foundation area. The foundation is designed essentially as a pile group to carry major part of the load, while some allowance is made for the load transmitted from the raft to the ground. This approach is more suitable for the small piled rafts (Hooper, 1974), i.e. where the pile support is required from a capacity point of view. The following is one of the case history of the performance of the piled raft foundations designed using this philosophy.

Cooke *et al.* (1981) presented the behaviour of piled raft foundation of a 16 storey block of flats in North London. The foundation consists of a heavily reinforced concrete raft 43.3 m x 19.2 m x 0.9 m and 351 bored cast in situ concrete piles. The piles are 0.45 m in diameter and 13 m long and placed in a rectangular grid pattern with centre-to-centre (c/c) spacing 1.6 m. The undrained shear strength of London clay varied from 100 kN/m² at a depth of 3 m to 260 kN/m² at a depth of 25 m. The value of the undrained shear strength of the clay immediately beneath the pile bases at a depth of 15.5 m below ground level was 190 kN/m². The full dead and live load was 155.6 MN. The piles were designed for a mean working load of 450 kN. Any

load carried by the raft was ignored. However, the monitored results indicated that at the end of construction about 25% of the applied load was carried by raft and 75% of the applied load was carried by piles. Further research and analysis indicated that the number of piles could have been reduced to less than 100, with only marginal increase in average settlement (Padfield & Sharrock, 1983; Randolph & Clancy, 1993).

Cooke et al. (1981) reported that:

1. At the end of construction, the ratio of settlement (short term) of the pile group to settlement of a single pile carrying a load equal to the mean of the loads carried by piles in the pile group was around 10.
2. Under full dead and live loads, the ratios of load carried by corner pile, edge pile and interior piles were 2.2:1.7:1.
3. Shear resistance mobilised by corner piles was higher than that of edge and internal piles. Shear resistance mobilised by internal piles was the least.
4. Base resistance mobilised by internal piles was greater than that of edge and corner piles. Base resistance mobilised by corner piles was the least.
5. Under full dead and live loads, the ratio of contact pressures at the corners of the raft to the contact pressures at central area was about 5.
6. As a result of consolidation the proportion of load carried by piles was increased and the proportion of load carried by raft was decreased. During early stages of construction about 55% of the total load was carried by the piles whereas after the building was occupied about 75% of total load was carried by piles.

As emphasized by Hooper (1974), Cooke (1986) and Randolph (1994), a relatively small numbers of piles are required to establish a region of reinforced soil with a high equivalent Young's modulus and to reduce the settlements. The addition of the piles further within the region yields little or no additional benefit towards reducing the settlements.

2.3.2 Creep piling approach

The creep piling approach was originally proposed by Hansbo & Källström (1983) for relatively soft cohesive soils, with the case histories described in detail by Hansbo & Jendeby (1983). The two principles behind this approach are:

1. Piles are designed to operate at a working load at which significant creep starts to occur. This is believed to begin at about 70-80% of the piles ultimate bearing capacity.
2. Sufficient numbers of piles are included to reduce the net contact pressure between raft and soil to below the preconsolidation pressure of the clay.

In this approach the foundation is designed essentially as a raft foundation, but the settlements are reduced in the same manner as suggested by Burland *et al.* (1977) by the inclusion of piles. One of the first building projects based on this principle of design was carried out in Gothenburg, Sweden (Hansbo & Källström, 1983). 'Creep piled' raft foundations have been constructed for a number of building projects in Sweden (Hansbo, 1984). The foundation costs are considerably reduced. The creep failure of piles can be determined on a theoretical basis from the undrained shear strength of clay. The settlement behaviour of two adjacent buildings in Gothenburg is shown in Figure 2.1.

The foundation soil is normally consolidated, highly plastic clay to a great depth. Both buildings have equal building loads of around 60 kN/m^2 . Building 1 was designed based on the conventional design approach with the factor of safety of 3 against pile failure. Building 2 was designed using the creep piling approach. For building 1, a total of 211 piles of 28 m length were used, where as for building 2 only 104 piles of 26 m length were used. Besides the reduction in foundation cost, the average settlement of building 2 was considerably smaller than the average settlement of building 1.

Figure 2.1 Settlements for two adjacent buildings constructed using the two design approaches for the piled foundations (after Hansbo, 1993)

2.3.3 Differential settlement control approach

The conventional and creep piling approaches described above adopt a uniform distribution of piles beneath the raft. In both cases, the primary aim is to limit the absolute settlement to an acceptable amount. Though, the differential settlements are reduced as a consequence of reduction in absolute settlements, a more direct approach is to design pile support in such a way as to minimise differential settlements, without necessarily reducing the average settlement. Figure 2.2 and

Figure 2.3 show schematically the principles behind the design of piles to reduce differential settlements (Burland *et al.*, 1977; Fleming *et al.*, 1992; Randolph, 1994; Horikoshi, 1995).

Figure 2.2 Central piles to reduce differential settlement (after Randolph, 1994)

When a uniformly distributed structural load is applied over a unpiled raft, then there will be a tendency for the unpiled raft to dish in the centre except for a very rigid unpiled raft. In such a case, a few piles added over the central region of the foundation, probably loaded to their ultimate capacity, will reduce the tendency, and thus minimise the differential settlements. The required pile support may be estimated by considering the 'ideal' contact pressure distribution that acts beneath a rigid raft, where the central contact pressure is approximately half the average applied pressure. Thus the central pile support is designed to carry about 40-70% of the total applied load.

Horikoshi (1995) has conducted an extensive parametric study to develop a rational design method for piled rafts to control differential settlements. His study showed that piled rafts could be designed for negligible differential settlements by including

a pile group over the central 16%-25% of the raft, with group stiffness approximately equal to that of raft alone.

Figure 2.3 Schematic design approach for differential settlement control (after Randolph, 1994)

Besides the three design approaches mentioned above, there is a more extreme version of creep piling (Poulos, 2001) which is not as commonly used as other design approaches. In this last category of design approach the full capacity of the piles is utilised, in which some or all of the piles operate at their ultimate load capacity. This gives rise to the concept of using piles primarily as settlement reducers. Burland (1995) has proposed a design method similar to this approach.

The simplified process of design proposed by Burland (1995) for piles to act primarily as settlement reducers may be outlined as follows:

1. Estimate the load-settlement relationship for the raft without piles (Figure 2.4). The design load Q_d gives a total settlement of δ_d .
2. Decide on an acceptable design settlement δ_a , which should include a margin of safety.
3. Q_R is the load carried by the raft corresponding to δ_a .
4. The design load P_0 in excess of P_I (i.e. $P_0 - P_I$) is assumed to be carried by the settlement reducing piles. The shaft resistance of these piles is assumed to be fully mobilised and therefore no factor of safety is applied. However, Burland suggests that a “mobilisation factor” of about 0.9 be applied to the conservative best estimate of ultimate shaft capacity, Q_s .
5. If the piles are located directly below the columns, which carry a load in excess of Q_s , the piled raft may be analysed as a raft on which reduced column loads act. At such columns, the reduced load Q_r is calculated from the actual column load Q as:

$$Q_r = Q - 0.9Q_s \quad \text{Eqn (2.1)}$$

The bending moments in the raft can then be obtained by analysing the piled raft as a raft subjected to the reduced loads Q_r .

2.4 Field Studies of Piled-Raft Foundations

Case 1: MesseTurm Tower in Frankfurt

Sommer *et al.* (1991) presented the performance of a piled raft supporting Europe's tallest building at that time, the 256 m MesseTurm Tower in Frankfurt. It is supported by a 6 m thick raft in the central portion and decreasing to 3 m at the edges. Figure 2.5 shows the plan of the building and location of piles. The piles were 1.3 m in diameter with lengths varying from 26.9 m to 34.9 m. A total of 64 piles were used. The spacing varied from 3.5 to 6 pile diameters. The piles were designed to develop their full geotechnical capacity and to carry about half of the design load. Figure 2.6 shows the load settlement behaviour of the tower (Tamaro,

1996). The total settlement of the building was about 11.5 cm by the end of year 1995, approximately 7 years after the commencement of construction.

Figure 2.4 Burland's simplified design concept (1995)

Figure 2.5 Piled raft foundation of MesseTurm Tower, Frankfurt, Germany (after Sommer *et al.*, 1991)

Figure 2.6 Settlements for Messeturm Tower, Frankfurt, Germany (after Tamaro, 1996)

Sommer et al. (1991) concluded that:

1. The settlements were reduced by 70% compared to the settlements of unpiled raft.
2. At the end of the construction the percentage of total applied load carried by piles was about 55%.
3. The contact pressure at the central region was higher than the contact pressure along the edges.

Case 2: Five storey office building in Urawa city, Japan

Yamashrita *et al.* (1994) investigated the piled raft foundation behaviour on stiff clay. The building is located in Urawa city, a suburb of Tokyo and around 15 m from the building on a piled raft discussed by the authors Yamashita and Kakurai (1991). The soil down to a depth of 6 m consists of stiff overconsolidated clay called Kanto Loam with an unconfined compressive strength of 150 kPa to 200 kPa. From a depth of 16 m to 19 m below the ground surface lies a medium to dense sand layer in which the toes of the piles were set. Under this layer, a number of clay and silt layers appear alternately to a depth of 42 m. The building is five-storey reinforced concrete construction and its plan dimensions are 24 m x 23 m. The design load is 47.5 MN and the available factor of safety against bearing capacity is 3. However, the settlement calculated using Steinbrenner's solutions for a multi-layered ground with a uniformly distributed load was 60 mm at the centre and 20 mm at the corner of the building. The average settlement and differential settlement were more than permissible values. Hence, a proposal was made to introduce piles for reducing differential settlement as suggested by Burland *et al.* (1977). A total of 20 piles were placed at each column position where the load from the superstructure was presumed to be concentrated. Figure 2.7 shows the layout of piles and monitoring positions. The number of piles required was calculated such that the ultimate bearing capacity of the pile group matches the required working load. In the case of this building the contribution to the bearing capacity from the raft was fully expected. Field observation made at nearby building showed that about 51% of load was carried by piles. This value was taken as the pile head load under

working condition. Table 2.1 shows the specification of the piles. The piles were constructed by inserting steel H-pile into a pre-augered borehole filled with mix-in-place soil cement. The length of the pile was 16 m and the spacing was 6.3 to 8.6 times pile diameter. Figure 2.8 shows the measured settlements.

Table 2.1 Specification of piles

Pile No.	Borehole dia. (m)	Size of steel-H (mm)
P1	0.80	414 x 405 x 18 x 28
P2	0.80	400 x 400 x 13 x 21
P3	0.70	350 x 350 x 12 x 19
P4	0.70	300 x 300 x 10 x 15

Figure 2.7 Foundation plan and monitoring positions (after Yamashita *et al.*, 1994)

Yamashita et al. (1994) concluded that:

1. At the time of the completion of the building, the settlements amounted to 10 - 20 mm. The maximum settlements are decreased by about 66% compared to the estimated values for unpiled raft.

2. At the time of completion of the project the piles carried about 49% of the total building load.
3. The contact pressures measured around the edge of the raft was higher than the contact pressures at central region of the raft. The contact pressure at corners was about 3-4 times higher than the contact pressures at the central region.

The piles in the central region of the raft mobilised higher loads compared to the piles along the periphery.

Figure 2.8 Measured settlements (after Yamashita *et al.*, 1994)

Case 4: TREPTOWERS of the Allianz Versicherungs AG in Berlin

Katzenbach *et al.* (1998) presented the load-settlement behaviour of a piled raft supporting the 121 m high office building TREPTOWERS of the Allianz Versicherungs AG in Berlin. For the first 3 m below ground level, the subsoil consists of fill, followed by loose sand and medium dense to dense sands in a depth of 40 m. The raft was located at 8 m below ground level in the area of the elevator pit and 4.5 m below ground level in the remaining parts of the building (Figure 2.9). This building was founded on a piled raft foundation with 54 bored, 0.88 m

diameter piles. Depending on the location of piles, the length of piles varied from 12.5 to 16 m. The base slab was 37.1 m x 37.1 m in plan and the thickness varies between 2 m and 3 m. Figure 2.10 shows the increase in load and settlement as construction was progressed.

Figure 2.9 Foundation- Plan and cross section (after Katzenbach *et al.*, 1998)

Figure 2.10 Variation of loads and settlements with time (after Katzenbach *et al.*, 1998)

Katzenbach et al. (1998) observed that:

1. At the end of construction the mean settlements are about 6.3 cm.
2. The corner pile which is of 16 m length carried around 1.3 times higher load compared to interior pile which is of 12.5 m length.
3. Further numerical analysis by Katzenbach *et al.* (1998) showed that the settlements of the piled raft foundation were decreased by about 57% compared to the unpiled raft foundation. The piles carried about 65% of total load.

Case 5: Circular steel tanks built for storage of sodium hydroxide in the area of port of Napoli

de Sanctis and Russo (2008) presented the performance of piled raft foundation adopted for circular steel tanks built for storage of sodium hydroxide in the area of port of Napoli. Figure 2.11 shows the plan of existing and new tanks. The existing tanks have diameters ranging from 7 to 10 m, heights from 10 to 12 m. The new tanks (numbers 11, 12, 13, 14 and 15) have diameters 10 and 12 m and a constant height of 15 m. The subsoil of the area consists of a deep silty sand deposit covered by a few meters of made ground. The water table is at around 2.5 m below the ground surface. The made ground located below the surface paving is around 5.25 m thick and consists of sand with fragments of bricks and rubble. This is underlain by about 20 m thick silty sand layer (upper sand layer) and is followed by another silty sand layer (lower sand layer). The lower sand is finer and looser than the upper sand. The SPT- N value increased from nearly zero at surface to 35-40 at a depth of 25m. The q_c profiles are more scattered and an average value of 15-20 MPa is reached at a depth of 20m. A conservative and constant value of the friction angle $\phi = 35^\circ$ was assumed based on the penetration tests.

For the 10 m and 12 m diameter tanks, the factor of safety against bearing capacity failure were respectively in the range of 8 and 9 for static load and 5 and 6 for pseudostatic earthquake loads. The settlement calculated based on the method proposed by Schmertmann *et al.* (1978) based on CPT values were around 90 mm and 105 mm respectively for the 10 m and 12 m diameter tanks. Similarly, the

settlement calculated based on Burland and Burbidge (1984) based on SPT values were around 157 mm and 180 mm respectively for the 10 m and 12 m diameter tanks. The allowable settlement is around 30 mm and allowable tilt is about 0.15%. Since, the calculated settlements are greater than allowable settlements, piles are considered to reduce the settlement to acceptable limits. Based on the site conditions continuous flight auger piles of 600 mm diameter and 11.3 m length are selected. The estimated allowable bearing capacity of the pile is about 810 kN with a factor of safety of 2.5. The traditional capacity based design required about 128 piles for the four tanks. However, to reduce settlement only 13 piles under each tank (52 piles for four tanks) have been placed.

Figure 2.11 Plan of existing and new tanks (after de Sanctis & Russo, 2008)

de Sanctis and Russo (2008) observed that:

1. The load carried by piles is about 53% and 52% respectively for the 10 m and 12 m diameter tanks at a loading of 90% of total capacity of the tanks.
2. The settlements are reduced by 90% and 73% respectively for the 10 m and 12 m diameter tanks.

3. In the case of 10 m diameter tank, the mobilised capacity of the piles is around 20% to 40% of estimated capacity of single pile. In the case of 12 m diameter tank the mobilised capacity of the piles is around 30% to 70% of estimated single pile capacity.
4. In both cases the piles along the edge mobilised higher loads compared to the interior piles.

Case 6: Eleven-storey office building in Aich prefecture, Japan

Figure 2.12 shows a schematic view of the eleven-storey office building and foundation with soil profile which is located in Aich prefecture, Japan. The building is a steel structure and is 80 x 43.5 m in plan. The height of the building is about 60.8 m above the ground surface. The subsoil consists of diluvial loose to medium sand with SPT *N*-values of 10–25 to a depth of 12 m from the ground surface, which is underlain by alternate layers of clayey soil, medium sand-and-gravel and dense sand with SPT *N*-values of 10–50 to a depth of 28 m. Below a depth of 28 m there lie dense to very dense sand-and-gravel layers. The groundwater table is about 17 m below the ground surface. The average contact pressure on raft foundation due to structural loading is about 145 kPa with local maximum contact pressure of 181 kPa. For this office building the maximum allowable inclination angle of the foundation is 1/2000 radian. The reinforced concrete raft was adopted for the foundation which was constructed about 3.0 m to 3.6 m below the ground surface. The raft was founded on loose sand with SPT *N*-values of about 10. The calculated differential settlement was more than allowable differential settlement. Hence, piles were introduced to reduce the differential settlement to allowable limits. A total of 40 numbers of cast-in-situ under-reamed concrete piles of 1.1 m to 1.5 m in shaft diameter and 1.4 m to 1.8 m in toe diameter were adopted. The length of the piles was about 26.9 m to 27.5 m. To confirm the design, the foundation settlement, axial forces mobilised by piles and contact earth pressures were monitored from the beginning of the construction and up to 32 months after the end of the construction. Figure 2.13 shows the foundation plan and location of monitoring devices.

Figure 2.12 Schematic view of eleven-storey office building in Aichi prefecture, Japan (after Yamashita *et al.*, 2009)

Figure 2.13 Layout of piles and with location of monitoring devices at eleven-storey office building in Aichi prefecture (after Yamashita *et al.*, 2009)

Yamashita et al. (2009) observed that:

1. The measured settlements were about 4-10 mm at 2.5 months before the end of construction. The observed maximum inclination angle was about $1/4500$ radian which is less than the allowable limit.
2. The ratio of load carried by piles to the net load on the tributary area was about 0.54 at the end of the construction and increased to 0.66 at 32 months after the end of the construction.

Case 7: Thirteen-storey hospital building in Osaka, Japan

Figure 2.14 shows a schematic view of the hospital building and foundation with soil profile. The building is about 51.3 m in height above the ground surface and measures about 55 m x 45 m in plan. The high rise part of the building is steel a framed structure while the low rise part and basement are reinforced concrete construction. The subsoil consists of loose sand and silty sand with SPT *N*-values of 5-10 up to a depth of 8 m below the ground surface. From a depth of 8 m to 21m there lie soft sandy silt and silty clay layers. Below a depth of 24 m there lie a diluvial very dense sand-and-gravel layer of SPT *N*-values of 50 or more. The ground water table is about 2.5 m below the ground surface. The hospital building has a basement and the foundation level is about 6.4 m below the ground surface. The average contact pressure is about 169 kPa at the high rise part and is about 114 kPa below the low rise part. For the low rise part, unpiled raft foundation was adopted as the pre-consolidation pressure was slightly larger than the average contact pressure. For the high rise part, piles were introduced to control the consolidation settlements and differential settlements. A total 17 numbers of bored pre-cast concrete piles in the inside of the high rise part with 198 steel H-piles built in the soil-cement diaphragm walls in the perimeter were adopted. The pile toes of the pre-cast concrete piles and the steel H-piles are embedded in the dense sand or sand-and-gravel layer below a depth of 24 m to ensure the end bearing resistance. The pre-cast concrete piles are of 0.8 m to 1.0 m in diameter and steel H-piles are of 0.4 m x 0.2 m in cross-sectional dimension. To verify the foundation design, foundations settlements, axial loads mobilised by the piles, contact earth pressure and pore-water pressures were monitored from the beginning of the construction to 28 months after the end of the construction. Figure 2.15 shows the foundation plan with the layout of piles and the instrumentation installed.

Yamashita et al. (2009) observed that:

1. The settlement of the foundation gradually increased until the completion of the building. After completion of the building the increase in the settlement of the building was relatively small. The observed maximum settlement was about 24 mm.

Figure 2.14 Schematic view of thirteen-storey office building in Osaka, Japan (after Yamashita *et al.*, 2009)

Figure 2.15 Layout of piles and monitoring devices at thirteen-storey office building in Osaka, Japan (after Yamashita *et al.*, 2009)

2. The measured pile head load increased gradually until the end of the construction and seemed to continue to increase slightly until 28 months after the end of the construction.
3. The ratio of the load carried by the piles to the total load on the tributary area was about 0.51 at the end of the construction and increased to 0.54 at 28 months after the end of the construction.
4. The measured pore-water pressure at the end of the construction was approximately equal to 50% of the observed contact pressure.

Case 8: Nineteen-storey residential tower in Kogoshima prefecture, Japan

The schematic view of the nineteen-storey residential tower and subsoil profile is shown in Figure 2.16. The residential tower is about 75.8 m in height. The subsoil profile consists of a loose to medium alluvial sand stratum with SPT *N*-value less than 10. This is underlain by diluvial medium to dense sand layers of SPT *N*-values of 20-50 to a depth of 105 m. The foundation level is about 3.2 m below the ground surface. The average contact pressure due to the structural loading is about 257 kPa. The calculated overall and differential settlements are more than permissible values. Hence, piles are adopted to reduce the settlements. It appeared that loose sand from a depth of 4-8 m to 18 m has a potential of liquefaction during earth quake. To cope with the liquefiable layer, a grid-form of soil-cement walls is constructed as shown in Figure 2.17. A total of 28 cast-in-situ under-reamed concrete piles of 1.2 to 1.3 m in shaft diameter and 1.8 m to 2.2 m in toe-diameter were adopted. The piles are of 63 m long and embedded in diluvial dense sand layer. To verify the design, foundation settlement, axial load carried by selected piles, contact earth pressure and pore-water pressures are monitored from the beginning of the construction and up to 15 months after the end of the construction.

Yamashita et al. (2009) observed that:

1. The foundation settlement gradually increased with construction process and amounted to 20 mm at the end of the construction. Thereafter, the settlement increased slightly and is about 22 mm at 15 months after the end of the construction.

Figure 2.16 Schematic view of nineteen-storey office building in Kogoshima, Japan (after Yamashita *et al.*, 2009)

Figure 2.17 Layout of piles and grid-form soil-cement walls with location of monitoring devices at nineteen-storey office building in Kogoshima, Japan (after Yamashita *et al.*, 2009)

2. The ratio of the load carried by piles to the net load on the tributary area was about 0.66 to 0.69 at the end of the construction and was increased to 0.69 to 0.77 at 15 months after the end of the construction. The ratio of load carried by pile toe to pile head load was about 0.42 both at the end of the construction and at 15 months after the end of the construction.
3. The measured pore-water pressures were small and almost drained conditions were observed throughout the construction process.

Case 9: Amuplaza building constructed using top-down construction method in Kagoshima city, Kyushu, Japan.

The commercial building has a seven storeys and a basement with a building area of 9000 m² and a maximum height of 45 m. A Ferris wheel was constructed on the roof of the building. The site investigation data showed that the stratification at the construction site was almost horizontal. The soil layers to a depth of 60 m are compressible sand or sandy silt. The building is shown in Figure 2.18. The SPT *N*-values are 10 to 20 to a depth of 50 m. The layer from a depth of 50 to 60 m is sand of sandy silt with SPT *N*-values varying from 20 to 42. Below a depth of 60 m, there exists gravel with SPT *N*-value greater than 50. The ground water table is about 3.0 m below the ground surface. The average contact stress of the unpiled raft is about 118 kPa. The design requirements for the building are the settlement is not to exceed 25 mm and the differential settlement is not to steeper than 1/1000. The bearing capacity of the unpiled raft is about 600 kPa, the available factor of safety against bearing capacity failure was about 5.

The calculated settlement was greater than permissible value and hence piles were used as settlement reducers. A total of 160 numbers of piles of 1.5 m to 2.0 m diameter and 20 m to 25 m long were adopted to reduce the settlements. The layout of the piles is shown in Figure 2.19. For this building top-down construction method was adopted. The cast-in-situ concrete piles were constructed from 6.5 m below the ground surface to required length. Steel columns of 7.5 m in length were attached to the pile heads. The ground water table was lowered from 3.0 m to 7.5 m below the ground surface. The excavation was carried out till the ground floor level.

Figure 2.18 Amuplaza building in Kagoshima city, Kyushu, Japan
(after Sonada *et al.*, 2009)

The steel beams were connected to the steel columns and ground floor slab was constructed. The excavation was then carried out till the formation level of the basement floor slab. The construction of the basement floor slab (raft) on the piles and the construction of superstructure proceeded simultaneously. Until the construction of the raft, the foundation was acted as a pile group and after the construction of the raft the foundation acted essentially as a piled raft. To verify the design the foundation settlements were monitored.

Sonada et al. (2009) observed that:

1. The foundation settlements were in the order of 4 mm to 14 mm much lesser than the allowable settlements.

2.5 Experimental Studies

Case 1: Model tests on unpiled rafts, free-standing pile groups and piled rafts

Cooke (1986) described the results of model tests on unpiled rafts, free-standing pile groups and piled rafts of various sizes. Model tests were conducted on remoulded London Clay having undrained shear strength between 5 kPa and 15 kPa. The model rafts were square steel plates of thickness between 13 mm and 25 mm. The size of the model rafts was 22-75 mm. The model piles were formed of brass rod of 3.2 mm in diameter and were 150 mm and 75 mm long.

Cooke (1986) reported that:

1. The ultimate capacities of the piled rafts and free-standing groups were similar for pile spacing that was closer than the critical spacing at which block behaviour of free-standing group occurs.
2. For pile spacing that was wider than the critical spacing, the ultimate capacity of the pile group was significantly increased by forming the raft on the clay surface.
3. The stiffness of the piled rafts compared to the corresponding pile group was at most 30% higher.
4. The distribution of load between the piles of piled raft foundation depends on structural loading and stiffness of structure foundation system.

Case 2: Centrifuge tests of model piled raft foundation

Horikoshi (1995) and Horikoshi & Randolph (1996), performed centrifuge tests of model piled raft foundation to examine the role of a central pile group in reducing differential settlement. A model scale of 1/100 was used with a nominal centrifugal acceleration of 100g. Kaolin clay was prepared in a rectangular strong box measuring 325 mm deep, 390 mm wide and 650 mm long. The clay was consolidated one-dimensionally in several steps, to a uniform vertical stress of 300 kPa.

The unit weight of the clay was 17.5 kN/m^3 , the coefficient of permeability was $4.4 \times 10^{-10} \text{ m/s}$ and the void ratio was 1.05 at the end of preconsolidation. The average shear strength of clay below the raft was about 40 kPa. The piles were made of tubular brass with an outside diameter of 3.15 mm and an inside diameter of 2.45 mm. The single pile capacity was approximately 32 N. The bearing capacity of the raft alone was about 3200 N. The maximum applied load was about 2100 N. Therefore the foundations had sufficient factor of safety against bearing capacity. Pile groups of 5 piles, 9 piles, 21 piles and 69 piles were modelled for piled raft models.

Horikoshi (1995) and Horikoshi & Randolph (1996) observed that:

1. The results of the single pile loading tests showed that the total capacity of the capped pile was significantly higher than that for the uncapped pile. It was also found that the pile itself had a higher bearing capacity for the capped case, which may be due to increased horizontal effective stress acting on the pile shaft.
2. The piled raft foundation supported by as few as nine piles reduced the differential settlement significantly. The differential settlement of this central piled raft was less than 30% of that unpiled raft.
3. This study showed that piled rafts could be designed for negligible differential settlements by including a pile group over the central 16-25% region/area of the raft, with the group stiffness approximately equal to that of raft alone. The pile group capacity should be around 40-70% of total applied load depending on the pile group area ratio and Poisson's ratio of the soil.

Case 3: Centrifuge tests on model piled raft foundations

Firovante (1998) presented the results of centrifuge tests on piled raft foundation. Tests were conducted on unpiled raft, raft with different numbers of piles and a solid block which simulated the upper limit of many piles that the group acts as block foundation. The geometrical scale of the model is 1/100, so an acceleration field of 100g was applied. The soil deposit is prepared in a cylindrical steel container with an internal diameter of 400 mm and a height of 440 mm. The model raft was a steel circular hollow plate of 66 mm in diameter and 15 mm in height. The model piles were aluminium tubes of 6 mm external diameter and 3.5 mm internal diameter. Load cells were placed both at pile head and pile tip to measure the pile head load and tip resistance. A layer of Toyoura sand approximately 250 mm thickness was placed at the bottom of tank by means of travelling sand spreader at constant height. Pontida silty clay with liquid limit 24 %, plasticity index 11%, and specific gravity 2.77 was used as a foundation soil. The clay was reconstituted from slurry with a moisture content of about 1.5 times the liquid limit, mixed under

vacuum for at least 24 hours, and poured in test container above the sand layer. The clay was then consolidated. The thickness of the consolidated clay layer was 190 mm and the OCR at the tip of the pile was approximately 1.3.

The model piles were installed on the clay layer at the end of preconsolidation, after boring with a special sampler of about 6 mm external diameter. A final thin layer of about 5 mm Toyoura sand was poured on the top surface of the clay to facilitate the contact between the raft and the pile heads. Finally a very rigid frame holding the loading system, the LVDT and the raft was mounted on the top of container. The model was then embarked in the centrifuge for the test. Figure 2.19 shows the load-settlement behaviour of unpiled raft and piled raft with different number of piles and also the load-settlement behaviour of block, which is an upper limit for settlement reduction.

Figure 2.19 Load settlement behaviour referred to prototype scale (after Fioravante, 1998)

Fioravante (1998) observed that:

1. The ultimate load mobilised by single pile belonging to a group was greater than the ultimate load mobilised by an isolated single pile.

2. In the case of rigid raft supported on piles, at smaller settlements piles along the edge carried higher loads compared to interior piles.

Case 4: Plane-strain experimental studies on model piled rafts

Cao (1998) has conducted plane-strain experimental studies to evaluate the performance of structurally disconnected piles as settlement reducers in sand. Mild steel plates of 220 mm x 440 mm and of different thickness (5, 10 and 25 mm) were used to simulate the model rafts having different stiffness. The model piles are made of aluminum tubes with a square section of 9.5 mm x 9.5 mm and 1 mm wall thickness. Piles of 350 mm and 500 mm lengths were used to study the effect of length of piles. The foundation soil is prepared in a reinforced concrete trench 1700 mm x 240 mm in plan and 800 mm deep. For simulating plane-strain conditions, the side wall friction was reduced by gluing 2 mm thick Perspex sheet to the inside surface of the concrete wall. A silicon-greased latex membrane 0.25 mm was then pasted on the Perspex sheet. Clean sand is used to prepare the model ground. The sand was placed in layers, each about 150 mm thick, by raining from a hopper. The relative density of sand so prepared was found to be 50%. A manual compactor was used to compact each sand layer immediately after raining, until sand attained a relative density of 70%. Different numbers of rows of piles and piles at different spacing were used to study the effect of pile arrangement.

Cao (1998) observed that:

1. The experimental results showed that using the structurally disconnected piles reduced the differential settlement and bending moment in the model rafts.
2. For a given pile group, an increase in pile length was found to be more effective in reducing the settlements.
3. With the increase of rigidity of the raft, the load transferred to the piles was reduced.
4. Negative skin friction was observed to affect the load transfer in the upper part of the piles.

5. In the case of 25 mm thick raft supported on 350 mm long piles, inner piles attracted more loads than the outer piles, whereas in the case of raft supported on 500 mm long piles outer piles attracted more loads compared to inner piles.

Case 5: Centrifuge tests on model piled raft foundations subjected to static vertical, horizontal and moment loading, and dynamic loading

Horikoshi *et al.* (2003a, b) and Matsumoto *et al.* (2004a, b) have presented results of a series of centrifuge tests on model piled raft foundations subjected different loading conditions viz. static horizontal, vertical and moment loading, and dynamic loads. Effect of the rigidity at the pile head connection was also studied.

Model tests presented by Horikoshi *et al.* (2003a) & Horikoshi *et al.* (2003b) were conducted on the same model piled rafts with different loading conditions. A centrifugal acceleration of 50g was applied to a 1/50 model. Aluminum plates of 80 mm x 120 mm and of 25 mm thickness were used to simulate the rigid rafts. An aluminum pipe with an outer diameter of 10 mm and an inner diameter of 8 mm were used to simulate the model piles. The pile toe was closed by using an aluminium plate. The length of the model piles was 180 mm. The model piles were instrumented with strain gauges to measure the axial loads and bending moments along the pile shaft. Air-pluviated dry Toyoura sand was used as foundation soil. The piles were kept in place and then dry sand was poured. Horikoshi *et al.* (2003a) have presented the performance of the model piled rafts subjected to static horizontal and vertical loads, whereas Horikoshi *et al.* (2003b) presented the performance of the model piled rafts subjected to dynamic loads.

Horikoshi et al. (2003a) observed that:

1. The stiffness and ultimate resistance of pile in the piled raft were different from those of single isolated pile.
2. By using piles and adopting rigid pile head connection, the ultimate horizontal resistance was increased in multi-fold compared to the unpiled raft.

3. The raft carried higher proportion of horizontal load initially at small displacements. With the increase of displacements the piles carried more load than the raft in the piled raft with rigid pile head connections.
4. During horizontal loading, the proportion of vertical loading carried by piles remained largely unchanged.

Horikoshi et al. (2003b) observed that:

1. Even during the dynamic loading, the proportion of horizontal load carried by each component was highly dependent on the horizontal displacement.
2. Even when the piled rafts were subjected to strong input motion, the change in the proportion of the vertical load carried by the piles was relatively small.
3. In reducing the horizontal acceleration, inclination and bending moments of the piles the contact between the raft base and soil surface played highly an important role.

Matsumoto *et al.* (2004a, b) have presented the results of 1-g tests conducted on the same model piled rafts and foundation soil used in the study presented by Horikoshi *et al.* (2003a, b). The 1-g model tests were conducted to study the behaviour of model piles rafts in sand subjected horizontal and moment loading. The model tests were carried-out with an aim to deduce the behaviour of the prototype and to compare the results obtained with that of results from centrifuge modelling presented by Horikoshi *et al.* (2003a, b). The model tests conducted were also focused in assessing the applicability of 1-g model tests to pile foundation problems. The results obtained from 1-g model tests confirmed the results obtained from the centrifuge modelling.

2.6 Analysis of Piled Rafts

The aim of any piled raft analysis is to find the load-settlement behaviour of the piled raft as a system. The critical information deduced from the analysis is overall settlements, differential settlements, load sharing between pile group and raft, load sharing among the piles, bending moment and torsional moments in the raft.

Various methods with varying degree of simplification ranging from fully simplified methods to more rigorous methods have been proposed to analyse piled raft foundations.

According to Poulos (2000) the analysis methods can broadly be grouped into the following three categories:

1. Simplified calculation methods
2. Approximate computer-based methods
3. More rigours computer-based methods

2.6.1 Simplified calculation methods

Simplified methods include those proposed by Poulos & Davis (1980), Randolph (1994) and Burland (1995). All these methods involve a number of simplifications in relation to the modelling of the foundation soil and the loading conditions on the raft. With these simplified methods only the ultimate bearing capacity and/or the settlement of the piled raft can be estimated by simple calculations. The available simplified analysis methods are briefly reviewed in the following sections.

Poulos-Davis-Randolph (PDR) method

According to this method the ultimate bearing capacity of the piled raft can be taken as the smaller of the following two values:

1. Sum of the ultimate capacities of the raft plus all the piles
2. Ultimate capacity of a block containing the piles and raft, plus that of the raft portion outside the periphery of the piles

$$P_u^* = P_u + Q_u^* \quad \text{Eqn (2.2)}$$

The load-settlement behaviour can be estimated by using the approach described by Poulos and Davis (1980). In this method it is assumed that for loading under undrained conditions, purely elastic conditions prevail up to the load at which the piles without cap would fail. After the piles have mobilised their ultimate capacities, it is assumed that any additional load is taken entirely by the raft and the additional settlement of the piled raft is due to the settlement of the raft only. Thus, the

undrained load-settlement curve of the piled raft system consists of two linear sections as shown in Figure 2.20.

However, a useful extension of this method can be made using the method outlined by Randolph (1994) for estimating the load sharing between raft and piles. According to Randolph, the stiffness of the piled raft foundation can be estimated as follows:

$$k_{pr} = \frac{k_{pg} + (1 - 2\alpha_{rp})k_r}{1 - \alpha_{rp}^2(k_r / k_p)} \quad \text{Eqn (2.3)}$$

Where,

k_{pr} = stiffness of piled raft; k_{pg} = stiffness of pile group

k_r = stiffness of raft alone;

α_{rp} = interaction factor between raft and pile group

Figure 2.20 Simplified load-settlement curve for preliminary analysis (after Poulos, 2001)

The raft stiffness can be estimated using the elastic solutions proposed by Fraser & Wardle (1976) or Myne & Poulos (1999). The pile group stiffness can also be estimated using the approaches proposed by Poulos & Davis (1980), Fleming *et al.* (1992) or Shen & Teh (2002).

The load distribution between piles and raft may be calculated using Eqn (2.4)

$$\frac{P_r}{P_r + P_p} = \frac{(1 - \alpha_{rp})k_r}{k_p + (1 - 2\alpha_{rp})k_r} \quad \text{Eqn (2.4)}$$

Where,

P_r = load carried by the raft

P_p = load carried by the pile group

$$\alpha_{rp} = 1 - \frac{\ln(r_c / r_o)}{\zeta}$$

r_c = average radius of pile cap, (corresponding to an area equal to the raft area divided by number of piles)

r_o = radius of pile

$$\zeta = \ln(r_m / r_o)$$

$$r_m = \{0.25 + \xi [2.5\rho(1 - \nu) - 0.25] \times L\}$$

$$\xi = E_{sl} / E_{sb}$$

$$\rho = E_{sav} / E_{sl}$$

ν = Poisson's ratio soil

L = pile length

E_{sl} = soil Young's modulus at level of pile tip

E_{sav} = average soil Young's modulus along pile shaft

E_{sb} = soil Young's modulus of bearing stratum below pile tip

The above set of equations can be used to develop a tri-linear load settlement curve, as shown in Figure 2.13. First, the stiffness of the piled raft can be computed from Eqn (2.3) for the number of piles being considered. The calculated stiffness is operative until pile capacity is fully mobilised. By assuming that the ultimate load mobilisation of all piles occurs simultaneously, the total load at which pile capacity is reached (P_2) is given by

$$P_2 = \frac{Q_u^*}{\left(1 - \frac{P_r}{P_t}\right)} \quad \text{Eqn (2.5)}$$

Where,

Q_u^* = ultimate load capacity of the piles in the group;

P_t = total applied load; P_r = load carried by the raft;

Beyond that point (point A in Figure 2.13), the stiffness of the system is that of the raft alone (k_r), and this holds until the ultimate load capacity of the piled raft foundation system is reached (point B in Figure 2.13). At that stage, the load settlement relationship becomes horizontal. The load-settlement curves for raft with different numbers of piles can be computed using above mentioned set of equations and simple spreadsheets or MATHCAD.

These simplified calculations methods are very useful for feasibility studies. However, for the final detailed design phase to assess the detailed settlement profile of the raft and to assess the optimum location of the piles and other required information for the design, approximate computer based methods or more rigorous computer methods have to be used.

2.6.2 Approximate computer-based methods

In approximate computer-based methods, the piled raft problem is simplified with realistic assumptions and approximations in relation to the representation of the soil, piles and raft and with regards to the interactions among them. The analyses are then conducted using one's own developed computer code or commercially available 2D or 3D FEM software. In approximate computer-based methods the soil can usually be represented using Winkler springs, Boussinesq springs, coupled springs, pseudo-coupled method, as an elastic continuum or elastic finite layer. The piles are represented as springs, rod elements or beam elements. The raft is represented as a strip, thin or thick plate. The interactions among the soil, pile and raft are obtained by integration of Mindlin's equation. The following are some of the available approximate computer-based methods for the analysis of piled rafts.

1. Methods employing a "strip on springs" approach in which soil is treated as an elastic continuum, the raft is represented by a series of strip footings and the piles are represented by springs of appropriate stiffness (e.g. Poulos, 1991).

2. Methods employing a “plate on springs” approach in which the raft is represented by a plate, the piles as springs and soil as an elastic continuum (e.g. Poulos, 1994a).
3. Methods in which raft is represented by a plate, piles and soil using linear/non linear springs (e.g. Russo, 1998; Kim *et al.*, 2001).
4. Methods that model raft by plate elements, piles as elastic beam elements or rod elements and soil as springs (e.g. Griffiths *et al.* 1991; Clancy & Randolph, 1993; Randolph & Clancy, 1993; Clancy & Randolph, 1996; Kitiyodom & Matsumoto, 2002; Kitiyodom & Matsumoto, 2003; Kitiyodom *et al.*, 2005).

The above methods are described in the following sections

Strip on springs approach

The method proposed by Poulos (1991) belongs to this category. In this approach, boundary elements are used to represent the structural components of the foundation i.e. raft and piles. The section of the raft is represented by a strip and is modelled as a beam, the supporting piles modelled by springs and the soil is represented as an elastic continuum (Figure 2.21). The four forms of interaction i.e. strip element to strip element, pile to pile, strip element to pile and pile to strip element are considered. The strip element to strip element interaction factors are determined using elastic solutions. The pile to pile interaction factors are determined from the approximate solutions proposed by Randolph (Fleming *et al.*, 1992). The influence of strip element on a pile is assumed to be given by the equation for the displacement of a strip element due to another strip element, multiplied by a reduction factor. The influence of a pile or pile group on a strip element is also computed from Randolph’s expressions for axial interaction factors. The effects of the parts of the raft outside the strip section being analysed are taken into account by computing the free-field soil settlements due to these parts. These settlements are then incorporated into the analysis and the strip section is analysed to obtain the settlements and moments due to the applied loading on that strip section.

The method has been implemented via a computer program GASP (Geotechnical Analysis of Strip with Piles). GASP can take account of soil non-linearity in an approximate manner by limiting the strip-soil contact pressures so as not to exceed the bearing capacity in compression or the raft uplift capacity in tension. The pile loads are limited so as not to exceed the compressive and uplift capacities of the piles. The individual pile load capacities are assumed to be the same as those for an isolated pile. The ultimate pile load capacities must be predetermined. The assumptions involved GASP analysis tends to be conservative.

Figure 2.21 Representation of piled strip problem (after Poulos, 1991)

The results obtained using GASP analyses were compared with the results from more accurate analyses (Brown *et al.*, 1975; and Hain, 1975). The comparisons indicate that the GASP can provide a reasonably satisfactory method of analysing the behaviour of piled strip foundations. The GASP analysis can also be used for analysing piled raft foundations by considering the influence of other sections as external applied loadings. The settlements obtained were found to be in reasonable

agreement. However, this approach did not provide accurate values of bending moments. The bending moments obtained by this approach due to the concentrated loads were under predicted, whereas the bending moments due to the uniform loading were over predicted. Furthermore, the method is not capable of giving consistent settlements at a point if strips in two directions through that point are analysed.

Plate on springs approach observation

As in the case of ‘strip on springs’ approach, in this method of analysis the soil is also represented as an elastic continuum and the piles are modelled as interacting springs but the raft is represented by an elastic plate instead of a strip. Some of the early approaches in this category (e.g. Hongladaromp *et al.*, 1973) neglected some of the components of interaction and gave pile-raft stiffness which was too large. Methods proposed by Poulos (1994a) and Hartmann & Jahn (2001) are some of the examples of this kind.

Poulos (1994a) modelled the raft as a thin plate and the piles as interacting springs of appropriate stiffness. Finite difference method was employed for the plate and various interactions are incorporated via elastic solutions. The method of analysis is outlined below.

1. The raft is discretised into a series of elements and nodes. Equations are developed for the vertical displacements of the raft itself due to bending under the action of the imposed loads and moments; and the contact pressures between the raft and the underlying soil.
2. Equations are developed for the vertical displacements of the soil at each node, in terms of the developed contact pressures between the raft and the soil and the externally imposed ‘free field’ soil moments.
3. At those elements where piles are located, the load carried by the piles is ‘smeared’ over the element, and the vertical displacement of the soil below the elements is obtained from consideration of the load-settlement characteristics of the pile.

4. Consideration is given to the four components of interaction i.e. interaction between raft elements, the interaction between piles, the influence of the raft elements on the piles, and the influence of the piles on the raft elements. The interaction factors are determined in the following way
 - i) Similar to the 'strip on springs' approach, the influence of raft elements on other raft elements is determined using elastic solutions.
 - ii) The pile to pile interaction factors are computed from the approximate solutions proposed by Randolph (Fleming *et al.*, 1992)
 - iii) The influence of raft element on a pile or pile group is assumed to be given by the equation for the displacement of a point at some characteristic depth ξL below the surface, due to another raft element. Based on the calibration with more accurate results it is observed the depth factor is about one-third.
 - iv) The influence of pile or pile group on a raft element is obtained from the pile-pile axial interaction factor.
5. The expressions for the vertical displacements of the raft and the soil are equated, and the resulting sets of simultaneous equations are solved for the unknown contact pressure and displacements.
6. The computed contact pressures are compared with the specified limiting values in tension and compression. At elements where the computed value exceeds the limiting value, the contact pressure is set equal to the limiting value, and the amended sets of equations are re-analysed.
7. From the computed contact pressures and displacements, the rotations, bending moments and shear forces in the raft are computed, together with the load carried by piles.

The results obtained using this approach are compared with the results from more accurate analyses (Hain & Lee, 1978; Selvadurai 1979; Kuwabara, 1989) and

measured behaviour (Thaher & Jessberger, 1991). Comparisons indicate that, the GARP analysis can provide acceptable results for problems involving elastic soil and pile response. However, the load capacity of the piles has to be limited to avoid under predicting the settlement and over predicting the amount of load carried by the piles.

Methods representing raft as a thin plate, soil and piles as springs

Russo (1998) modelled the raft as a thin plate and the piles as interacting non-linear springs. Soil is modelled as an elastic layer using linear springs. Figure 2.22 shows the basic features of the model. In this analysis it is assumed that the interaction between raft and the soil or the piles is purely vertical. Hence, only axial stiffness of the springs is used in the analysis. The soil spring stiffness is deduced from the closed form solutions for the settlement of a uniformly loaded rectangular area at the boundary of a homogeneous elastic half-space. The method is capable of analysing both force and moment loadings. The numerical procedure is implemented via a computer program NAPRA (Non-linear Analysis of Piled Rafts). The rectangular rafts with any flexibility and any numbers of piles and in any pattern can be analysed using NAPRA. The analysis results of the raft supported on elastic half-space using this method has been checked with well-known benchmark solutions (Fraser & Wardle, 1976) and found reasonable agreement. The analysis results using NAPRA for a piled raft with 8 x 8 pile group are compared with results obtained by using a full BEM solution (Kuwabara, 1989). The general trend of the results is similar. However, the present method tends to over-predict the total load carried by piles for slenderness ratio (L/d) > 7.5 and under-predict the total load carried by the piles for slenderness ratios < 7.5. The results obtained using the methods are also comparable with the measured behaviour (Thaher & Jessberger, 1991a, 1991b and Horikoshi, 1995). The settlements of the foundation obtained from NAPRA are found to be in good agreement with measured results. However, the load sharing between the piles and raft obtained from NAPRA is not in good agreement with measured values.

Figure 2.22 Basic features of the model for piled raft (after Russo, 1998)

Anagnostopoulos & Geogiadis (1998) modelled the raft as a slab and piles as supporting vertical springs which represent the axial pile stiffness. Figure 2.23 shows the basic features of the model. The analysis is conducted using SAP 90 (finite element computer code). Part of the building load is supported by the pile springs, while the remaining part is directly applied to the ground through several uniformly loaded disks. Four interaction factors among the various foundation elements, determined from simple analytical expressions, were incorporated in the analysis. The distribution of the load among various foundation elements is achieved through an iterative procedure. Initially the loads are distributed arbitrarily, and then the pile settlements and stiffness are determined considering the interaction factors among various foundation elements. Subsequently, the circular disc on vertical springs is analysed and new settlements and load distributions are determined. The iterations continued until the final load distribution, settlements, raft bending moments and shear forces are obtained.

Kim *et al.* (2001) modelled raft as a thin plate, piles as coupled springs and soil using Winkler springs. The FE model for the pile raft foundation is shown in Figure 2.24. The stiffness of piles is obtained from the approximate solution proposed by Randolph & Wroth (1979). The Winkler spring constant is obtained using modulus of subgrade reaction. The raft is modelled as a plate using Mindlin theory and discretised by isoparametric finite elements. The stiffness equation for piled raft is obtained by the finite element method. In this approach, the interaction between the

piles was considered approximately in the coupled springs as proposed by Randolph & Wroth (1979). The interaction between piles and raft is neglected as the Winkler spring model is used. Therefore, this approach may overestimate the stiffness of the piled raft foundation. This proposed model with an optimisation scheme has been used to arrive at optimal pile arrangement for minimising differential settlements.

Figure 2.23 Typical piled raft and its numerical modelling (after Anagnostopoulos & Geogiadis, 1998)

Figure 2.24 FE model of a piled raft (Kim *et al.*, 2001)

Yamashita *et al.* (1998) represented the piled raft problem slightly differently. The raft was modelled using beam elements and bending plate elements. Figure 2.25 shows the analytical model for the raft-soil pile system. The piles and soil were represented by interacting springs. Soil non-linearity was considered in the interacting springs. The load-displacement relation of the soil was modelled as a bi-linear function. The procedure of analysis consists of two stages. In the first stage the interacting pile and soil spring constants were expressed in terms of reaction forces and displacements. In the second step, the spring constants obtained in the first step were used to calculate total settlement, bending moment and shear force of the raft. The reaction forces and the settlements thus obtained were used to calculate the new values of spring constants. These iterative calculations were continued until the spring constants of piles converge within certain value of tolerance.

Methods representing raft as thin plate, piles using rod elements or as elastic beams and soil as springs

Griffiths *et al.* (1991), Clancy & Randolph (1993), Randolph & Clancy (1993) and Clancy & Randolph (1996) have modelled raft as a thin plate, piles with rod elements and soil as springs. This approach is based on the analysis method proposed by Chow (1986) and Chow (1987) for pile groups. Figure 2.26 shows the representation of piled raft problem.

Figure 2.25 Analytical model for raft-soil pile system by Yamashita *et al.* (after Yamashita *et al.*, 1998)

In this approach 1-D rod finite elements are used to represent each axially loaded pile. The soil response is coupled at each pile node and is modelled by discrete load transfer (t - z) springs. The analytical method proposed by Randolph and Wroth (1979) is adopted for calculating the gradient of the load transfer springs. The interaction between the piles through the soil is calculated using Mindlin's elastic solution. In the present method no numerical integration of Mindlin's solution over pile surface is required since the forces are considered to be at each pile node. The use of load transfer springs eliminates the need to calculate interactions between nodes of the same pile. A separate raft analysis has been developed using 2-D thin plate bending finite elements. An equivalent soil spring is attached at each raft node. The spring response is calculated using analytical solution proposed by Giroud (1968) for the average settlement under a uniformly loaded rectangular area. The contributing area of the raft to each node is calculated by adding the area of each raft element to which the node is attached and divided by four (since each raft element has four nodes). The centre of the contributing area may not necessarily at node. Therefore, the centres of the contribution areas are determined and these centroidal points are then used to calculate the interaction between raft nodes through soil. Mindlin's equation is again adopted to calculate interaction factors. The two analyses are combined by attaching piles to the raft via common nodes at the connecting points. The vertical degrees of freedom are linked, resulting in only axial load being transmitted to the piles. The interaction between raft nodes and pile nodes is calculated using Mindlin's equation. Though the soil, pile and raft responses and the interactions among them are approximated yet rigorous treatment has been given to the piled raft problem compared to the other approaches described in Section 2.6.3. However, to analyse larger pile groups using this approach, a large amount of computational power is required. Hence for practical purposes the analysis is limited to groups of around 50 piles or less.

1. One-dimensional pile element
2. Ground resistance at each pile node represented by non-linear 'T-Z' springs
3. Two-dimensional plate-bending finite element raft mesh
4. Ground resistance at each raft node represented by an equivalent spring
5. Pile-soil-pile interaction effects calculated between pairs of nodes using Mindlin's equation
6. Pile-soil-raft interaction
7. Raft-soil-raft interaction

Figure 2.26 Numerical representation of piled raft (after Clancy and Randolph, 1993)

Kitiyodom & Matsumoto (2002) modelled the raft as a thin plate; the piles, instead of rod elements, as elastic beams and the soil as springs. This model is similar to the model proposed by Clancy & Randolph (1993) except that two additional soil springs in horizontal plane are attached at each node of the piles and the raft to account for the lateral resistance and bending of the piles and shear resistance between raft base and soil surface. The analytical model is incorporated in a computer program PRAB (Piled Raft Analysis with Batter piles). Figure 2.27 shows the features of the model. The raft and pile element models are combined via the nodes at the pile heads. The various interactions are modelled using Mindlin's solutions for both vertical and lateral forces. This analysis is applicable only to homogeneous soil. Kitiyodom & Matsumoto (2003) extended the analysis to non-homogeneous soils by modifying the expressions for soil spring constants. Averaged Young's modulus of different layers was used in the approximation of the interactions.

Figure 2.27 Plate-beam-spring modeling of piled raft
(after Kitiyodom & Matsumoto, 2002)

2.6.3 More rigorous computer-based methods

The more rigorous computer-based methods include the following broad approaches:

1. Boundary element method in which both the raft and the piles are discretised and elastic theory is used to analyse the system (e.g. Butterfield & Banerjee, 1971b; Brown & Wiesner, 1975; Kuwabara, 1989; Sinha, 1997).
2. Methods combining finite layer theory to analyse the layered soil and finite element theory to analyse the rafts (Hain & Lee, 1978; Franke *et al.*, 1994; Zhang & Small, 1991; Ta & Small, 1996; Ta & Small, 1997; Ta & Small, 1998; Zhang & Small, 2000; Small & Zhang, 2000).

3. Simplified finite element analyses, usually involving the representation of the foundation system as an axi-symmetric problem (Hooper, 1974) or a plane strain problem (Desai, 1974; Cao, 1998; Prakoso & Kulhawy, 2001).
4. Three-dimensional finite element analyses (Zhuang *et al.*, 1991; Lee, 1993; Wang, 1996; Smith & Wang, 1998; Katzenbach *et al.*, 1998; Katzenbach *et al.* 2005; de Sanctis, 2001; Liang *et al.* 2003; Reul, 2000; Reul & Randolph, 2002; Reul & Randolph, 2003; Reul & Randolph, 2004; Reul, 2004; Maharaj & Gandhi, 2004).

Boundary element methods

Brown & Weiesner (1975) analysed piled strip footings by idealising the problem as a uniformly loaded strip footing supported by a number of identical, equally spaced piles located along the centre-line of the footing. Figure 2.28 shows the basic features of the model. The soil was modelled as an isotropic homogeneous elastic half-space. The pressure between the soil and the footing was assumed to consist of a number of zones of uniform pressure extending over the full width of the footing. The force between a pile and the footing was treated as concentrated upward force at the centre of a reaction zone (Figure 2.28 (c)).

The interaction stresses on the exterior surface of a pile were assumed to consist of zones of uniform vertical stresses, composed of one zone of normal stress at the base of the pile and a number of zones of uniform shear stress along the pile shaft. The assumed pressure distributions were ideal pressure distributions. These pressure distributions may not exactly represent those which would occur if the soil were a perfectly linear elastic continuum. However, the settlements were calculated along the centre line of the footing where the settlements are primarily controlled by pile settlements not by the pressure distribution beneath the footing. The interaction effects were determined by the integration of Mindlin's expression. Relationship between the stresses and the relative displacements of the zone centres in the piles and footing were determined by considering piles as simple compression members and by the use of simple bending theory for the footing. The soil, pile or footing

vertical displacements were equated at the centre of each zone, and the resulting equations plus vertical equilibrium equations are used to determine the interaction stress. The stresses thus obtained were used to determine settlements and bending moments.

The obtained settlements were the settlements along the longitudinal centre line of footing. Significant reduction in settlement and differential settlements was observed due to the installations of piles beneath a uniformly loaded strip footing. When the footing is moderately or very stiff only small reductions in positive moments were observed. In reducing the settlements, long and/or stiff piles were seen to be more effective than short and/or compressive piles. However, due to the assumptions with regards to the pressure distributions and interactions, these results have to be validated in the light of field or experimental results.

Kuwabara (1989) conducted an analysis of piled raft foundations in homogeneous isotropic elastic half-space based on elastic theory. This method of analysis is an extension of the procedure for single piles first proposed by Mattes & Poulos (1969). The analysis procedure adopted is outlined in the following section.

The pile group consists of N identical elastic piles in which each pile has a length L , diameter d , Young's modulus E_p , and is divided into n_s shaft elements and n_b base elements. The surrounding soil is assumed to be a homogeneous isotropic half-space having Young's modulus E_s and Poisson's ratio ν_s . The vertical displacement of the soil adjacent to the piles and the raft due to the stress $\{p\}$

$$\{\rho_s\} = \frac{d}{E_s} [I_s] \{p\} \quad \text{Eqn (2.6)}$$

Where:

- $\{\rho_s\}$ = soil displacement vector
- $\{p\}$ = vertical stress vector on pile-soil or raft-soil interface
- $[I_s]$ = vertical displacement influence matrix

Figure 2.28 Analytical model of typical piled raft (after Brown & Wiesner, 1975)

The pile displacement is the sum of the pile tip displacement and the compression of the pile between the point considered and the tip due to the vertical shear stress on the pile. The displacement of the foundation and the adjacent soil are equal when slip or local yield on the interface does not occur. Invoking the vertical equilibrium condition of the total system allows all the stresses on the interface and the vertical displacement of the raft to be solved. It is observed that, the percentage of load carried by the raft is about 20 – 40% of the total applied load for $L/d < 50$, $s/d < 10$ in an undrained condition. The vertical distribution of load in pile is little affected by the presence of the raft except in the upper part of the shaft where the load is

reduced by the raft. The contact pressure is relatively uniform in the inside area surrounded by the piles and this contact pressure is much smaller compared with the contact pressure in the overhang area of the raft.

Methods combining finite layer method for the soil and finite element method for the raft

The finite layer method can be used to analyse the problems where the soil is made up of number of layers. In the finite layer method, the stresses and the displacements are often expressed as their Fourier or Hankel transformations. Hain & Lee (1978) developed an analysis procedure for raft having any stiffness supported on a group of compressible/incompressible piles by combining the finite element method for the raft and finite layer method for the soil. The supporting soil was assumed to be an elastic homogeneous/non-homogeneous material with a modulus which increases with depth. The raft was composed of rectangular plate bending elements. The pile group-supporting soil system was treated as a pile reinforced continuum and modelled using the Mindlin's Equation. The pile-pile, pile-soil, soil-pile interactions factors were defined as follows.

Pile-pile interaction factor, α was defined as

$$\alpha_p = \frac{\text{additional displacement due to unit load on adjacent pile}}{\text{displacement of pile due to unit load}} \quad \text{Eqn (2.7)}$$

Pile –soil interaction factor, β_p was defined as

$$\beta_p = \frac{\text{additional displacement of a pile due to unit surface pressure}}{\text{displacement of pile due to unit load}} \quad \text{Eqn (2.8)}$$

Soil-pile interaction factor, β_s was defined as

$$\beta_s = \frac{\text{additional displacement of the surface due to unit pile load}}{\text{displacement of the surface due to unit load}} \quad \text{Eqn (2.9)}$$

Using pile-pile, pile-soil, and soil-pile interaction factors the stiffness equations for the soil-pile group system are constructed. By invoking the compatibility and equilibrium conditions between soil-pile group system and the raft, a set of stiffness

equations representing the raft-supporting soil-pile group system were obtained. In doing this two assumptions were made. One, vertical forces only are transmitted from the raft to the pile head and second, each pile occupies the whole of the constant pressure zone around a particular node. The first assumption implies that the connection between the raft and the pile is a sliding ball joint. In this analysis the lateral forces and moments were neglected. Second assumption implies that the raft and the supporting soil are only connected at nodal points. Figure 2.29 shows the discretisation of the problem and the load distributions. In this analysis, the developed pile loads were restricted to ultimate pile loads using a load cut-off procedure.

Figure 2.29 Arrangement of soil elements and pile elements for compatibility with the raft (after Hain & Lee, 1978)

The ultimate pile loads were calculated using the shaft resistance and base resistance. In the load cut-off procedure, the piles that reach the ultimate loads were deleted from compatibility equations and their loads were held constant at the ultimate values in the equilibrium equations. Thus, any excess load was redistributed to the adjacent piles.

$$\{W\} = [K_R' + K_{fp}] \{w\} \quad \text{Eqn (2.10)}$$

Where $\{W\}$ = effective transverse loads on the raft
 K_R' = raft stiffness in terms of transverse displacements
 K_{fp} = supporting soil-pile group stiffness matrix
 $\{w\}$ = transverse settlements

The settlements are calculated using Eqn (2.9). The bending moments and pile loads and soil surface reaction pressures are obtained from the calculated settlements. From the analysis results, it was clear that relatively few piles are required to reduce the settlements. The reduction in settlements becomes more effective with an increase in pile stiffness and pile length.

Ta & Small (1996) analysed a piled raft system using a similar approach as Hain & Lee (1978). The raft was modelled using the finite element method and treated as thin elastic plate, and soil-pile group system was modelled using the finite layer method. The raft was divided into a number of rectangular elements (Figure 2.30) each having 4 nodes and 16 degrees of freedom. The contact pressures between the raft and the soil and between the raft and pile heads are assumed to be uniform blocks of pressure, which act over each element in the raft. A system of simultaneous equations for raft displacement, pile displacement and soil displacement were derived. Imposing the compatibility and equilibrium conditions, the resulting simultaneous equations are solved for the unknown quantities. The results obtained using this approach for uniformly loaded unpiled circular rafts resting on deep elastic homogeneous soils were compared with Brown (1969) solutions obtained using rigorous elastic theory. The results are in reasonably close agreement with Brown's solutions. A piled raft system with 9 piles was analysed and the results were compared with the results obtained by Kuwabara (1989). Overall, the loads predicted in the piles by both the methods were reasonably close. However this approach gives slightly higher loads along the lower sections of the pile shaft and slightly smaller loads at the top of the shaft were predicted.

Figure 2.30 Force considered to act on the piles and on the soil (after Ta & Small, 1996)

Simplified finite element analyses, usually involving the representation of the foundation system as an axi-symmetric or plane strain problem

Prakoso & Kulhawy (2001) analysed piled rafts using plane strain finite element modelling. The geotechnical finite element code PLAXIS was used for their study. The plain strain model involves the fundamental simplification of condensing a finite size piled raft to a strip piled raft. In this case, an in-plane row of piles were simplified into an equivalent plane pile. The equivalent Young's modulus for the plane strain pile was calculated using the following equation.

$$E_{eq} = \frac{n_{p-row} A_p E_p}{L_r d} \quad \text{Eqn (2.11)}$$

Where,

n_{p-row} = number of piles in a row; A_p = pile cross-sectional area;

E_p = Young's modulus of the pile; d = pile diameter;

L_r = in-plane raft length

The side resistance of the equivalent plane strain piles was modified to have same compression capacity as that of the in-plane row of piles they represent and the effect of tip resistance was assumed to be insignificant for deep piles. The side resistance was modelled using interface elements and the equivalent side resistance for an interface element at a given depth on each side was calculated using the following equation.

$$f_{s-eq} = \frac{n_{p-row} A_s f_s}{2L_r} \quad \text{Eqn (2.12)}$$

Where,

A_s = shaft area per unit depth; and f_s = side resistance for unit depth.

The reliability of the plane strain model was examined by comparing with the results obtained using 3D finite element model (Wang, 1996). The plane strain model tends to overestimate the displacements for most of the cases considered. The order of the overestimation was about 5-25%. The bending moments obtained using plane strain model were similar to those across the raft in a 3D model. The plane strain differential settlement (centre-edge) was about 2/3 of the centre-corner differential settlement of the 3D model.

However, as pointed out by de Sanctis & Russo (2001), in elasto-plastic analysis the stress path followed by a soil element in plane strain and constrained between rows of piles is different from that followed by a soil element in a real pile group. And hence, the results obtained using plane strain modelling should be used with caution.

Three dimensional finite element analyses

The interaction of piles, raft and soil in a piled raft foundation is an extremely complex problem. Previous attempts in rigorous analysis (e.g. Butterfield & Banerjee, 1971a; Desai, 1974; Hooper, 1974; Brown & Wiesner, 1975; Hain & Lee, 1978; Kuwabara 1989; Franke *et al*, 1994; Ta & Small, 1996; Sinha 1997; Russo & Viggiani, 1998; Zhang & Small 2000; Prakoso & Kulhawy, 2001) inevitably involve simplification with regards to the modelling of soil and interface response. In the boundary element methods (Brown & Weiesner, 1975; Kuwabara, 1989) or

in combined finite element and finite layer methods (Hain & Lee, 1978; Ta & Small, 1996; Zhang & Small, 2000) the pile-soil response was assumed to be elastic. It is also well known that modelling the raft with thin plates may lead to higher stresses in the soil. Mindlin's solution of a point load in an elastic half space formed the basis of most pile group analysis reported in the literature. Though these highly idealised solutions provide some insight into the pile group response, they do not provide a sufficient basis for a thorough understanding of the load transfer mechanism. However, with the availability of more sophisticated software, the need for simplification is reduced. Yet for practical purposes, simplifications are required as the cost of analysis and computing time for more rigorous three-dimensional modelling are high. But for establishing certain benchmark solutions, which are to be compared with the simplified methods and to evaluate the available simplified methods, three-dimensional modelling is required as the piled raft is in reality a three dimensional non-linear problem. The following are some of the analyses which were carried out using three dimensional FE modelling.

Ketzenbach *et al.* (1998) back analysed a piled raft foundation (TREP TOWERS in Berlin) using three dimensional finite element model and compared the results with the filed measurements. The soil and piles were modelled using isoparametric finite elements and the raft was modelled with shell elements. At the boundaries infinite elements were used. Taking advantage of symmetry only a quarter model needed to be modelled. The piles and raft material behaviour was simulated using linear elastic material model, whereas the soil material behaviour was simulated using an elasto-plastic model. The calculated load-settlement behaviour of the piled raft was in good agreement with the measurements. The pile resistance was over estimated.

Katzenbach *et al.* (2005) back analysed the performance of the piled raft foundation supporting the 110 m high EUROTHEUM in Frankfurt. The soil and piles are represented by first-order solid continuum elements of brick and wedge shape. The raft was modelled using first-order shell elements. The contact behaviour between soil and foundation (raft and piles) was modelled using thin solid continuum elements with specified soil material parameters. The three-dimensional mesh was generated using the pre-processor PATRAN and the FE simulations were carried out using Abaqus. The three phase (solid, liquid and gas) soil medium was

simplified to single phase medium (solid). Consolidation effects were ignored. The soil material behaviour was simulated using elasto-plastic cap model available in Abaqus.

Reul & Randolph (2003) presented the comparative study of three dimensional finite element analysis results with in-situ measurements. The soil and piles are represented by first-order solid finite elements of hexahedron (brick) and triangular prism (wedge) shape. The raft was modelled using first-order shell elements of square and triangular shape coupled with reduced integration. The soil above the foundation level is replaced by its self weight. The circular piles were replaced by square piles with the same shaft circumference. The non-linear material behaviour of the soil was modelled by a cap model that consists of three yield segments: the pressure-dependent, perfectly plastic shear failure surface, the compression cap yield surface and the transition yield surface. The contact between the structure and soil was modelled as perfectly rough. At the contact between the soil and raft, and between soil and pile, thin solid continuum elements were used instead of special interface elements. The material behaviour in the contact area was prescribed with the material behaviour of the soil. This study was concerned with the verification of in-situ measurements of a full-scale structure. The comparisons of overall settlements, differential settlements show reasonably good agreement with the measured values. However, the proportion of load carried by the piles obtained from FE analysis were larger than the values derived from measurements in the field.

Using the same 3D finite element approach, Reul & Randolph (2004) conducted a parametric study of 259 different pile raft configurations. The parameters used in their study were pile positions, the numbers of piles, pile length, raft-soil stiffness ratio and the load distribution on the raft. The parametric study indicates that the optimum design of the piled raft foundations mainly depends on the subsoil conditions, the load configuration and load level. The following conclusions were made from the parametric study:

1. In general, for the same total pile length smaller average settlements were obtained with longer pile rather than large number of short piles.

2. The differential settlements are more sensitive to the raft-soil stiffness ratio and load configuration. For a raft under uniform loading or core-edge loading, the differential settlements can be most effectively reduced by placing the piles only under the central area of the raft.

Using the same 3D finite element approach Reul (2004) has explored the bearing behaviour of piled rafts and made the following conclusions:

1. Due to pile-raft interaction the skin friction was shown to increase with increase in load or settlement.
2. The resistance and equivalent spring stiffness of a pile in piled raft depend on the position of the pile and the settlement of the superstructure.
3. The ultimate capacity of the piles in piled raft may be greater or less than the ultimate capacity of single individual pile depending upon the location of pile and number piles in the piled raft.

Maharaj and Gandhi (2004) presented the results of undrained, three dimensional finite element analyses of a raft and piled raft foundations in clay that have been loaded until failure. The raft, piles and soil have been discretised as 8-noded brick elements. The soil has been modelled using Drucker – Prager material model. The piles and raft are modelled using linear isotropic material model and the analysis is carried out using ANSYS general finite element software. The rafts with 16 m x 16 m plan dimensions and thickness 0.1 m, 1 m and 2 m are analysed. The load settlement curves of plain rafts indicated that, for lower soil stiffness all the three rafts acted as rigid rafts. But with the increase of the stiffness the raft with 0.1 m thickness showed flexible raft behaviour whereas the rafts with 1 m thickness and 2 m thickness showed rigid behaviour. For a plain raft at lower loading intensity the contact stresses are a minimum in the central portion and a maximum near the corner of the raft. However, with the increase of the loading intensity the contact stresses at different points below the raft become almost the same. In a piled raft foundation the corner pile carried maximum load. The edge piles carried higher load compared to centre pile but smaller load compared to corner pile. The centre pile carried smallest amount of load. The corner piles reach its ultimate capacity at

smallest amount of settlement, followed by edge pile, and the corner piles reach their ultimate capacity at higher settlement.

de Sancties and Mandolini (2006) have explored the bearing capacity of piled rafts on soft clay soils by means of 3D finite element analyses using Abaqus version 6.2. The soil was assumed to be elasto-plastic with a Tresca yield surface. The pile soil interface was modelled using very thin elements of thickness equal to around 0.1 times the diameter of the pile and having the same properties as the adjacent soil. The raft was assumed to be rigid and a displacement controlled analyses was adopted. The loading was simulated by applying uniform increments of vertical displacements to the soil surface.

Liu *et al.*, (1994) and Borel (2001) suggested modifying Eq. (2.12) for ultimate capacity of piled raft as follows

$$P_u^* = \alpha_{UR} \cdot P_u + \alpha_G \cdot Q_u^* \quad \text{Eqn (2.13)}$$

de Sancties & Mandolini (2006) have explored the values of α_{UR} and α_G . Based on their study it was found that α_G is approximately equal to 1 for the geometrical configurations analysed. The value of α_{UR} varies in the range of 0.4 to 1.

2.7 Summary

The generalisation of the results obtained from the studies on piled rafts to all the possible cases in the field is an extremely difficult task. However, in general the pile support in a raft is provided for two reasons: either to increase the load carrying capacity of the foundation or to reduce the settlements of the raft. In situations where raft foundation alone has ample factor of safety against bearing capacity failure but settles excessively under the working load, piles can be introduced to reduce the settlements. The concept of piled raft foundation is presented in Section 2.2 in which piles are used to reduce the settlements. The most favourable condition for using piles to reduce the settlements is when the foundation soil provides sufficient factor of safety against bearing capacity failure, and the raft alone results in excessive average/differential settlements. This is usually associated with a foundation soil which is stiff clay or dense sand.

The three design approaches in practice for designing piled rafts are conventional approach, creep piling approach and differential control approach. In all these design approaches, piles are used to reduce the settlements; but these are designed essentially as load carrying members thereby the settlements are reduced. However, the settlement based approach could provide a more economical solution than load based design approach. In-situ studies, experimental studies and analytical studies (Hooper, 1974; Cooke *et al.*, 1981; Hansbo & Källström, 1983; Randolph, 1994; Poulos, 2000; Poulos, 2001) reveal that relatively small number of piles are required to bring the settlements to acceptable limits and adding piles further provides little or no additional benefit.

Burland (1995) proposed a new method for designing piles primarily to reduce settlements. In this method, it is assumed that the unpiled raft has sufficient factor of safety against bearing capacity failure and piles are introduced only to reduce settlements. And, piles are assumed to operate under their ultimate capacities. However, uncertainties still exist with regard to the behaviour of piles at working loads on the piled raft. It is unclear whether all piles really mobilise their ultimate capacity at permissible settlement of the raft. Based on the results obtained from previous studies (Cooke *et al.*, 1981; Yamashita & Kakurai, 1991; Yamashita *et al.*, 1994; Cao, 1998; Firovante, 1998; Katzenbach *et al.*, 1998; de Sanctis & Russo, 2008), it is evident that the axial loads carried by the piles located at different positions under the raft are different at a given settlement of raft. Moreover, the proportion of the ultimate capacity of single individual pile actually mobilised at permissible settlement by piles in piled raft located at different positions is not well-understood. The study of how settlements are being modified with the introduction of limited number of piles of different sizes under the raft deserves further investigation. A clear understanding of the mechanism by which piles aid in reducing raft settlement would go a long way towards more economical use of piled raft foundation.

Numerous approximate methods for the analysis of piled rafts were proposed by different authors (Griffiths *et al.*, 1991; Poulos, 1991; Clancy & Randolph, 1993; Randolph & Clancy, 1993; Poulos, 1994a; Clancy & Randolph, 1996; Russo, 1998; Yamashita, 1998; Kim *et al.*, 2001; Kitiyodom & Matsumoto, 2002; Kitiyodom &

Matsumoto, 2003; Kitiyodom *et al.*, 2005) with varying degree of simplification. These studies signify the need for an approximate method which is simple and yet provides reliable solutions. The practicing engineers usually adopt methods representing soil and piles as springs as rigorous analysis using 3D finite element simulations are time consuming and expensive. For the approximate methods, the soil spring stiffness is usually obtained using modulus of subgrade reaction or deduced from the closed form solutions for the settlement of a uniformly loaded rectangular area at the boundary of a homogeneous elastic half-space. Such elastic solutions are highly idealised and may not truly reflect the interface response in a real piled raft. A study into the interface contact pressure response and the mobilization of pile capacity with settlement of the raft would aid in a better understanding of piled-raft behaviour, particularly when the piles were introduced to mitigate settlement.

Chapter 3

Experimental Set-up and Test Procedures

3.1 Introduction

The primary objective of this study is to investigate how piles would modify the settlement of a raft. The context is that the raft would be able to carry the imposed load safely, but the ensuing settlement is not acceptable. Piles used in this context may be termed settlement reducing piles to emphasise the specific purpose of the piles. Since bearing capacity failure is not a concern even for the unpiled raft, the capacity of the piles can be fully utilised. That is, the factor of safety of the piles with regards to geotechnical resistance can be assumed to be one.

For measurement accuracy, the model foundation soil should be constituted in such a way that appreciable settlement would ensue under the applied load. It is also necessary to ensure that the properties of the foundation soil could be repeatedly reproduced for the number of model tests planned. Uniformity of the soil properties is another requirement. In this study, reconstituted kaolin was used as the foundation soil.

Only one size of the model raft was used in the study, but length and width of the piles were varied. The number of piles, and the length and width of the piles would be varied from test to test to study the effect of these parameters on the performance of the raft.

The following sections provide a comprehensive description of the various components used in the model tests. The instrumentation deployed is discussed in detail.

3.2 Foundation Soil

The foundation soil was prepared from kaolin. To ensure consistency, 400 bags of kaolin powder each weighing 25 kg was purchased in a single order. The basic properties of the kaolin were determined on 10 samples collected randomly from 10 different bags. The results showed that the variation in the basic properties of the soils sampled in this way was insignificant. The process of preparing the foundation soil would be presented in a latter section.

3.3 Model Rafts

The model raft was fabricated using solid aluminium plate. It consisted of two square plates 300 mm in width (Photo 3.1 & Figure 3.1). The top plate was 5 mm thick and the bottom plate 10 mm thick. During test, the two plates were rigidly connected together with bolts and nuts so that they behaved essentially as a monolithic unit. This design was selected for two purposes. Firstly, the grooves in the connecting faces of the two plates provided pathways for the instrumentation cables. Secondly, such an arrangement would allow for the piles to be threaded through the preformed openings in the bottom plate and be rigidly fixed in place. The top plate would provide an even top surface for the loading to be applied. Different bottom plates would be used depending on the number of piles deployed.

Photo 3.1 Model raft

Figure 3.1 Model raft

3.4 Model Piles

Various factors were taken into consideration when designing the model piles. The pile sizes obviously would be constrained by the size of the raft, the number of piles and pile spacing. The above factors controlled the dimension of the model pile cross-sections.

The pile length was constrained by the thickness of the foundation soil. It was decided that only two pile lengths would be used. The length was chosen such that one was longer, and the other shorter, than the width of the raft. The model raft was 300 mm wide, so the pile length was set at 200 mm and 400 mm respectively.

Strain gauges would be installed on the model pile to measure the changes of axial load along the pile. To facilitate the installation of the strain gauges on the piles, it was decided to use square hollow aluminium sections. The flat surface allowed the strain gauges to be installed more easily. The central hollow provided a passage for threading the instrumentation cables.

Further consideration based on estimated pile load bearing capacity and pile stiffness led to the final choice of four model pile types each constructed of square aluminium hollow section with a wall thickness of 2 mm. The four pile types were based on a combination of 2 sectional dimensions and 2 pile length. The piles were designated as follows (Table 3.1).

Table 3.1 Model piles dimensions and designations

S. No	Model Pile	Dimension (mm)	Cross sectional area (mm ²)
1	B2L2	20 x 20 by 200	144
2	B2L4	20 x 20 by 400	144
3	B3L2	30 x 30 by 200	224
4	B3L4	30 x 30 by 400	224

Properties of model piles:

Wall thickness = 2 mm; Young's modulus = 6.85×10^7 kPa

Poisson's ratio = 0.35

The first two characters in the pile designation indicate the pile width: B2 and B3 referred to a width of 20 mm and 30 mm, respectively. The last two letters referred to the pile length. Hence, L2 and L4 referred to a pile length of 200 mm and 400 mm, respectively.

Model pile fabrication and connection to raft

In practice the piles would be rigidly connected to the raft. Hence, due consideration was given to the connection between the raft and pile. Photo 3.2 and Figure 3.2 show the schematics of the model pile used in this study. It was formed from 2 channel sections. The two channel sections were assembled together using small aluminium plates and screwed together to form a pile of square cross-section. At one end of the pile a tip, pyramidal in shape, was attached to aid driving of the piles into the soil. At the other end of the pile, a threaded circular section of about 10 mm was welded. The threaded portion of the pile head was inserted into the opening made in the bottom plate of the raft.

A specially designed cap as shown in Figure 3.2 was used to tie the pile with the raft. The square portion of the pile head, which was wider than the opening of the raft, acted as a stopper and helped in providing rigid connection between the raft and pile. The strain gauges were glued on to the inner surface of the channel sections to avoid possible damage during installation.

Figure 3.2 Schematics of model pile

3.5 Equipment and Measurement System used in Model Tests

The model test required the use of many components. The primary ones being the equipment needed to prepare the model ground and the loading system. The equipment used to prepare model ground included the slurry mixer and the consolidation tank. The loading system comprised a hydraulic servo mechanism with a 'rigid' reaction frame. The third major component was the data logging

system. In addition, several miscellaneous equipment would be deployed in the course of the study to assist in pile installation and post-test soil sampling.

The technical specifications of each of these components would be described in the following sections.

Following is the list of test apparatus used in the model tests:

- a) slurry mixer and consolidation tank
- b) loading system for applying central point load
- c) instrumentation for measuring various parameters including
 - applied consolidation pressure
 - settlement
 - applied load on the model foundation
 - axial load distribution along the pile shaft
 - raft-soil contact pressure
- d) data acquisition system, and
- e) other accessories which include:
 - mats to guide pile installation
 - square tube with cutting edges to make borehole
 - portable vane shear apparatus for in-situ probing

3.5.1 Slurry mixer and consolidation tank

The capacity of the slurry mixer was 0.06 m^3 . The mixing time adopted was 30 minutes. The consolidation tank was constructed of Grade-316 stainless steel designed to resist a pressure of up to 700 kPa. The internal diameter of the tank was 1 m and the wall thickness was 20 mm. Figure 3.3 shows the cross-section of the consolidometer.

The consolidometer was an assembly comprising 3 cylinders, a base plate, a piston and a top plate. The total height of the consolidometer was 1.4 m. The bottom two cylinders were 0.4 m in height and the top cylinder was 0.6 m high. The inner surface of the cylinders was honed to minimise friction. The thickness of the base plate, piston and top plate was 30 mm each and were made of stainless steel. At the centre of the base plate and at the centre of the piston a recess of approximately 60

mm in diameter and 15 mm height was provided. Porous stones of appropriate size were installed in this recess to provide drainage.

Figure 3.3 Schematics of consolidometer and instrumentation

At the centre of the base plate (i.e. at the centre of recess) was a hole 10 mm in diameter for drainage during the consolidation process. The piston was connected to a 40 mm diameter shaft made of stainless steel. A 10 mm coaxial hole in the shaft provided a drainage path for water. Two openings with valves were provided on the top plate, one for the inlet of compressed air and other to act as an emergency exit for air. The base plate, cylinders and top plate were connected together with 12 numbers of M16 bolts around the perimeter. The water and air tightness between different parts of consolidometer were achieved using rubber O-rings.

3.5.2 Loading system

In this study, loading on the piled raft was applied as a central point load. Figure 3.4 and Photo 3.3 show the details of a central point load setup. In this setup, the load from the actuator was transferred through a stainless steel roller to the raft. The setup consisted of a 25 mm diameter mild steel rod (loading rod) at the tip of this rod, a small groove was made so that it could sit snugly on the roller. A similar groove was provided on the raft, for the roller to seat properly on the raft. The head of this rod was welded to a 10 mm thick plate (load cell base plate), 100 mm in diameter. The load cell was placed on this plate. The plate and loading rod assembly was suspended using three mild steel rods with lock nuts from the plate which was affixed to the actuator ram. These steel rods would allow free downward movement of actuator ram. When the ram travelled downwards the load cell was compressed and the load was recorded. The load cell details are discussed in the following sections.

Figure 3.4 Schematic diagram showing the set-up for applying central point load on raft

Photo 3.3 Set-up of the loading system for applying central point load on raft

3.5.3 Instrumentation systems

Accurate monitoring of various key parameters was essential to achieve the objectives of the study. The key measurements obtained during the tests included the consolidation pressure, applied load on the raft, settlement and deformation of the raft, raft-soil contact pressures and axial load distribution along the pile shaft. The details of instruments and sensors used to obtain the various measurements are as follows.

Consolidation pressure

In the preparation of the foundation soil, it was crucial to measure the consolidation pressure as the clay slurry consolidation was in progress. The applied consolidation pressure was monitored using a BUDENBERG test gauge (Photo 3.4). The maximum range of this pressure gauge was 300 kPa. The practical accuracy of this pressure gauge was $\pm 0.25\%$. The pressure gauge would ensure that the consolidation pressure was maintained throughout the sample preparation process.

Displacement measurement during model ground preparation

During preparation of the model ground, monitoring of consolidation settlement was required to estimate the degree and rate of consolidation. It was estimated that the consolidation settlement would vary between 400 mm to 600 mm, depending on the applied air pressure and initial height of the clay slurry. TML displacement transducer model CDP-100 was used to monitor the consolidation settlement during sample preparation. The maximum range of measurement of the transducer was 100 mm with an accuracy of about $\pm 0.25\%$. Since the consolidation settlement was greater than the range of transducer, the displacement transducer needed to be reset during consolidation.

Displacement measurements for piled raft settlement and deformation

When a central point load was applied on the piled raft, the piled raft might not settle evenly. Even with care, slight eccentricity of the applied load might not be entirely avoidable. This coupled with any variation in the foundation soil properties would cause the raft to tilt. The settlement and deformation of the raft during the load tests were monitored at seven locations on the raft. These locations are shown in Figure 3.5 and Photo 3.5. The maximum settlement of the raft foundation was estimated to be between 30 to 50 mm. The displacements were monitored using TML displacement transducers model CDP-100. The maximum range of the transducer was 100 mm with an accuracy of 0.01mm. Before deployment, all the displacement transducers were calibrated using Mitutoyo calibrator.

Photo 3.4 Test gauge to monitor consolidation pressure

Figure 3.5 Monitoring positions of displacement transducers on the raft

Photo 3.5 Monitoring positions of displacement transducers on the raft

Measurement of applied load on raft foundation

The model raft was installed on the foundation soil formed within the consolidometer. During load test, the entire assembly was moved and positioned within the loading frame equipped with a hydraulic servo mechanism. The servo mechanism would allow the displacement rate to be controlled. The load applied onto the raft was monitored using a TML KC-10M load cell (Figure 3.4 and Photo 3.5). The maximum range of the load cell was 100 kN.

Before deployment in the test, the load cell was calibrated using a compression testing machine with an internal load cell. A predetermined increment of load was applied and the corresponding output from the load cell was recorded. The readings were taken using the same data logger used in this study. For greater consistency, the same channel on the data logger was used for the load cell both during calibration and throughout the testing programme. This was to avoid errors that might arise as calibration using different channels might give rise to different calibration coefficients. The location of this load cell is shown in Figure 3.4 and Photo 3.5.

Measurement of axial load distribution along pile

The load transmitted to each pile and the variation of axial load along the pile could be deduced from the axial strain at different elevations of the pile. In this study, the model piles were strain-gauged at three locations. The strain gauge positions on pile are shown in Figure 3.7. The top strain gauge meant for monitoring pile head load was installed at 2 cm below the pile head. Another strain gauge was placed 2 cm from the pile tip. The third was located at the mid-level of the pile.

TML electrical strain gauges of type FLA-10-23 were adopted. The gauge length of these strain gauges was 10 mm. The stain gauges were bonded to the pile surface using compatible adhesive supplied by the manufacturer. For calibration, the individual single pile loading tests were conducted with load cell to measure the applied load, and by comparing the strain at pile head and applied load, the calibration coefficients were obtained.

Figure 3.6 Monitoring positions of stain gauges on model pile

Measurement of interface contact pressure between raft and soil

The relationship between the interface contact pressure distribution beneath the raft and the corresponding raft settlement (i.e. modulus of subgrade reaction) is a key parameter in the analysis of rafts using the soil spring apparatus. The interface contact pressure is unlikely to be uniform throughout the raft-soil interface. Knowledge of how the interface contact pressure varies across the raft is essential for estimating the contribution of raft bearing pressure to the total piled raft bearing resistance. It also renders possible a more accurate assessment of the load sharing between the raft and pile group.

The contact pressure distribution beneath the raft was monitored using KYOWA miniature pressure transducers. These sensors were circular in shape and were approximately 5 mm in diameter and 2 mm in thickness. The sensors were carefully glued to the raft using the special glue supplied with the sensors. The change caused by the sensors on the interface behaviour was insignificant because of their small size. The maximum range of the sensor measurement was approximately 2 MPa with an accuracy of $\pm 0.5\%$. A total of seven miniature pressure transducers were installed in the raft. The locations of these transducers are shown in Photo 3.6 and Figure 3.7.

Photo 3.6 Locations of miniature pressure transducers on the underside of the raft

Figure 3.7 Locations of miniature pressure transducers on the underside of the raft

3.5.4 Data acquisition system

All the sensors and monitoring instrument, were TML strain type sensors. (i.e. electrical strain gauges, displacement transducers, miniature pressure transducers). A TML TDS-302 data logger was used to capture the data from the TML strain type sensors.

The TDS-302 data logger facilitated the measurement of static strain, pressure and load. The data logger comes with a built-in switching box with 10-channels. Each channel could monitor one sensor at a time. In this study, approximately 15 to 40 channels were required depending on the model piled raft foundation setup. To handle the data-logging needs of multiple sensors, an automatic switching box ASW-30B with an additional 30 channels was used. The data logger was connected to a personal computer for data capture and storage via the Visual-log static measurement software version 2.16.0.

3.5.5 Other ancillary equipments

In addition to the major pieces of equipment described above, several auxiliary components and equipment were also deployed to facilitate the conduct of the model piled raft tests. This included accessories such as the guides used to ensure verticality of the piles during installation and the square stainless steel tubes with bevelled cutting edges to create the pre-formed holes for the piles. In addition, portable vane shear apparatus was used for post-test in-situ probing of the foundation soil. The following paragraphs provide a brief description of these ancillary components and equipments.

Pile installation guide

The pile group installation guide consisted of longitudinal bars and a set of cross plates (Figure 3.8). The number of cross plates in a set was determined by the number of piles in the piled raft, square notches were cut in the cross plates to receive the piles. The cross plates were connected to the longitudinal bars using small screws. As shown in Figure 3.8, two similar assemblies of longitudinal bars and cross plates were deployed during pile installation. The horizontality of the cross plates assembly was checked using spirit level. The verticality of the piles was ensured by the alignment of the square openings in the installation guide (Figure 3.8). The guide assembly was placed above the foundation soil without touching the soil surface. The setup was thoroughly checked to ensure verticality during the formation of the pre-formed holes for the piles and during pile installation.

Device for pre-formed hole for pile installation

Two square tubes made of stainless steel, with sharp bevelled cutting edges were used to make the boreholes. The cutting edges were bevelled inside to minimize disturbance to the soil outside. Square tube 10mm and 20mm in cross-section were used to drill the boreholes for the square piles 20mm and 30mm in width respectively. In other words, the pre-formed holes were 10mm smaller than the actual pile cross-section.

Figure 3.8 Pile Installation Guide used to install piles vertically for 4 piles raft

Portable vane shear apparatus

A simple portable vane shear apparatus was used for post-test in-situ probing of the soil. The vane shear apparatus was supplied with a calibrated scale. The vane was inserted to the required depth in the foundation soil, the wing was rotated and the torque required to shear the soil was recorded. Using the calibrated scale, the corresponding undrained shear strength (c_u) was obtained.

3.6 Typical Procedures in the Model Piled Raft Test

Three main stages may be identified for each piled raft test. These are:

- preparation of the foundation soil made from kaolin.
- load test on model piled raft – including installation of piles and raft, setting up the instrumentation, load application and data logging.
- post test probing of the soil.

The steps involved in these 3 major stages are described in the following section.

3.6.1 Preparation of foundation soil

The preparation and consolidation of clay can be divided into three steps

- mixing of kaolin slurry
- setting up the consolidometer
- consolidation of kaolin slurry

Mixing of kaolin slurry

The slurry method was used to prepare the foundation soil. The quantity of slurry required was calculated based on the required height of the foundation soil and the consolidation properties of the slurry. To ensure uniformity in the resulting soil specimen and to prevent air entrapment, the initial water content was set at 2 to 2.5 times liquid limit (LL) as recommended by McManus & Kulhawy (1991). An initial trial preparation was carried out to check the consistency and uniformity in mixing and sample preparation. Specimens obtained from the trial sample were subjected to a wide range of basic soil tests. Based on this initial trial, it was identified that a water content of 135%, which was equivalent to 2.2 times liquid limit (LL) was optimum for the slurry. From the initial trial, it had also been found that mixing 25 kg kaolin powder at 135% of water content and consolidating it under 200 kPa pressure would produce a foundation soil model 1 m in diameter and 29 mm thick. The resulting soil sample had undrained shear strength c_u of 40 kPa. The water content at the end of consolidation was about 47%.

Given the length of the model pile, it was decided that the height of the foundation soil should be at least 700 mm thick for all the loading tests. Based on the trial test results, around 600 kg of kaolin powder would be required for each model ground. In the laboratory, the capacity of the available slurry mixer was 0.06 m^3 , much smaller than the volume of the clay slurry required. Hence, mixing of kaolin powder was carried out in 24 batches of 25 kg each. Mixing a 25 kg batch would take about 30 minutes. In total, about 12 to 13 hours time was required to mix the entire quantity. Therefore, each batch was separately mixed and stored in small containers until 24 mixed batches had been obtained. Prior to pouring the slurry into the consolidometer, each batch was again re-mixed for a short duration of about 5 minutes before being transferred into the consolidometer.

Setting up the consolidometer

A porous stone of appropriate diameter was placed in the recess at the centre of the base plate. A very thin layer of hydraulic oil was applied on the inside surfaces of the cylinders using brush to minimise the frictional effects between soil and consolidometer. The bottom cylinder was placed on the base plate with rubber O-rings between them and tightly connected using bolts and nuts. Two layers of saturated filter cloth were placed on the base plate. On the top of the saturated filter cloth, filter papers 320 mm in diameter were placed in concentric rings with the inner ring of filter paper slightly overlapping the outer ring. The filter papers were placed in this manner to prevent slippage during pouring of the kaolin slurry. The filter papers and filter cloth were kept saturated to avoid any displacement. All these procedures were repeated for the preparation of foundation soil for each model test.

Placement and consolidation of kaolin slurry

The slurry, which was prepared as described in previous section, was poured into the consolidometer. Initially, only the bottom cylinder was installed on the base plate, and the first 350 mm height of the slurry was poured manually with care to prevent any displacement of the bottom filter paper and filter cloth. The slurry was poured at the centre and allowed to spread outwards towards the periphery. Once the slurry had reached a height of 400 mm, the middle and top cylinders were

placed on top of the bottom cylinder with O-rings in between them, and tightly bolted. The remaining slurry was then pumped into the consolidometer using an electrical pump. At every stage of the preparation, adequate care was taken to prevent entrapment of air. The entire process of re-mixing and pouring the slurry into consolidometer took about 15 hours at one stretch. The total height of the slurry in the consolidation was about 1300 mm to 1400 mm.

After the entire quantity of the slurry had been transferred into the consolidometer, the slurry was allowed to undergo self-weight consolidation for about 10 hours. After the period of self-weight consolidation, the supernatant water was removed without undue disturbance to the settled clay, leaving about 10 mm height of water on the surface. Similar to the filter paper placed at the bottom of the tank, a layer of filter paper was also placed on top of the slurry. The water on top of the slurry was retained because it was easy to place filter paper and filter cloth without disturbing the settled clay. Two layers of filter cloth were placed on the top of the filter paper. Considering the size of the soil sample, two layers of filter cloth were used. The porous stone was placed in the recess provided at the underside of the piston. The O-rings on the bottom face of the piston were cleaned of any dirt, and a thin layer of grease was applied to facilitate the smooth sliding of piston inside the tank. The piston and the top plate assembly were then engaged. The top plate was then connected to the top cylinder with O-ring in between them.

To advance the piston, 10 kPa air pressure was applied while keeping the top drainage valve open until the piston just seated on the surface of the slurry. This was signalled by the appearance of water draining through the top drainage valve. Once the piston had contacted the surface of the slurry, the drainage valves were closed and the consolidation pressure applied. The displacement transducer to measure consolidation settlement (Figure 3.3) was set in place and connected to the data logger.

Before opening the drainage valves, the consolidometer was thoroughly checked for leakage. The drainage valves were then opened and the changes in settlement were monitored by the automatic data logger. It took about 6 days for the completion of primary consolidation. In all tests, the duration for the application of consolidation

pressure was kept the same (8 days), so as to maintain consistency in the properties of the consolidated soil sample.

At the end of primary consolidation, the top plate and piston were removed along with the top cylinder. The 2 remaining cylinders of consolidometer with the consolidated soil was then transferred to the loading frame in preparation for the model piled raft test.

3.6.2 Load test on model piled raft

When the consolidation of the clay slurry was in progress, the raft and the piles would be readied for installation. The miniature pressure transducers were glued to the bottom side of the raft at locations shown in Figure 3.7. Strain gauges were affixed to the selected key piles (Figure 3.6).

Once the foundation soil had been fully consolidated, the soil sample contained within the lower cylinders of the consolidometer was transferred to the loading frame. The centre of the soil sample and the position of the raft and piles were marked on the soil surface. Pre-formed holes, 10 mm smaller in size than the pile width, were made using the stainless steel tube as discussed in Section 3.5.5. The pile installation guide assembly (Figure 3.8) was used to ensure verticality. The raft and piles were assembled together and the entire assembly was jacked in using the actuator. The mats described in Section 3.5.5 were used to guide the installation of the piles (Figure 3.9 and Photo 3.7). The entire piled-raft assembly was pushed into the soil at a rate of 15 mm/min using the actuator. The installation guide was removed when the pile group had penetrated at least 150 mm into the soil. After removal of the guide, the piled-raft assembly was jacked until the raft touched the surface of the foundation soil.

Once the piled raft has been installed, the LVDTs were positioned at the marked locations as shown in Figure 3.5. The device for applying central point load was connected to the actuator ram with the load cell securely put in place. All the sensors were connected to the data logger which was in turn hooked up to the personal computer.

Figure 3.9 Schematic diagram of piles installation process

Photo 3.7 Pile installation process

It took a full day to complete the installation of the model piled raft on the consolidated foundation soil, and readying the data logging system. Once the instrumentation had been verified, the piston driven by the actuator would begin to apply loading on the piled raft. The test was displacement controlled using a penetration rate of 0.05 mm/min.

To derive the maximum return from the huge investment incurred in the sample preparation, the piled raft was tested in 3 load-unload cycles. The primary focus of this study was on the first loading path, but the data obtained from the load-unload cycles could provide additional information which would contribute to better understanding of the piled raft behaviour.

In the first and second loading cycles, the raft was loaded to about 12 mm raft displacement. In the third loading cycle, the raft was loaded up to a displacement of 50 mm, on equivalent to 16.7% raft width. A time interval of approximately 16-18 hours was allowed to elapse between the complete unloading of the preceding load cycle and the start of the following load cycle. During the load cycles, the applied load, the raft displacement and deformation, the raft-soil interface contact pressures, and the axial loads carried by the piles were recorded. After completion of the

loading tests, portable vane shear device was used to determine the in-situ c_u of the foundation soils. The raft and piles were removed carefully. Soil samples were collected at various locations and at various depths for determining the water content and undrained shear strength.

3.6.3 Post test probing of the foundation soil

The characterisation of the foundation soil in each model test was an essential part of this study for proper interpretation of the results. Immediately after testing, portable vane shear apparatus were used to determine the in-situ undrained shear strength. In total, vane shear tests were conducted at 8 locations and at different depths outside the footprint of the piled raft and the undrained shear strength values were recorded. Soil samples were also collected randomly at various locations and depths for water content determination. Specimens were also retrieved for triaxial testings.

3.7 Testing Programme

The testing programme involved in this study could be divided into four phases. These are preparatory tests, tests on unpiled rafts, individual load tests on single piles, and the main test programme involving various piled rafts comprising piles in different configurations.

Preparatory Tests

- 1) The reliability of the experimental data was crucially dependent on the accuracy and performance of the measuring equipment and the sensors deployed. These equipment and sensors needed to be properly calibrated to ensure that the readings are accurate. All the measuring instruments, viz. displacement transducers, miniature pressure transducers, and load cell were calibrated both at the start and end of the testing programme. The difference in the calibration coefficients obtained at the start and end of testing programme was found to be insignificant. The calibration charts are presented in Appendix A.

- 2) As described in Section 3.2, 10 soil samples were collected from different bags and a series of standard laboratory tests were carried out to determine the basic index properties of kaolin clay, that is, grain size distribution, liquid limit (LL), plastic limit (PL), and specific gravity (G_s). These properties are presented in Section 3.8.
- 3) Before the start of main testing programme, two sets of soil samples were prepared in a small consolidometer under a consolidation pressure of 200 kPa and 250 kPa. A series of unconsolidated-undrained (UU) triaxial tests, consolidated-undrained (CU) triaxial tests and oedometer tests were conducted on these samples to characterise the soil.
- 4) To study the behaviour of the unpiled rafts, two tests were performed using the same unpiled raft in foundation soils prepared using two consolidation pressures of 200 kPa and 250 kPa respectively. These two tests served to check the consistency and reliability of preparation method. It also provided an opportunity to assess the feasibility of making preformed holes for tests on single piles and piled raft specimens. The tests on the unpiled raft led to the final adoption using 200 kPa consolidation pressure in preparing foundation soil for the main test programme.
- 5) To characterise the behaviour of individual single piles, loading tests were conducted on all four model pile types. The test on one of the single piles was repeated to address the issue of repeatability of test results.
- 6) A total of 9 model piled rafts tests were conducted in this study. Four of these involved 4-pile piled rafts. Another four tests were conducted on 9-pile piled rafts. Finally, a test on 16-pile piled raft was performed. For ease of reference, the model piled rafts tests were also assigned a 6 letter notation. The first 2 letters referred to the number of piles. The second 2 denote the pile width and the third pair of letters denotes the pile length. Hence, 4PB3L2 would denote a 4-pile piled raft in which the piles are 30 mm wide by 200 mm long. The designation of the 9 piled raft specimens are summarised in Table 3.2.

Table 3.2 Summary of experiments on piled raft specimens

Specimen	No of Piles	Width of piles (mm)	Length Piles (mm)	Spacing
Raft dimension 300 mm x 300 mm				
4PB2L2	4	20	200	7.5B
4PB3L2	4	30	200	5.0B
4PB2L4	4	20	400	7.5B
4PB3L4	4	30	400	5.0B
9PB2L2	9	20	200	5.0B
9PB3L2	9	30	300	3.3B
9PB2L4	9	20	400	5.0B
9PB3L4	9	30	400	3.3B
16PB3L2	16	30	200	2.5B

3.8 Index Properties of Kaolin Clay

As mentioned in Section 3.2, 10 kaolin clay samples were collected randomly from different bags and index properties were determined on these samples. The test results showed that the variation was insignificant. The index properties of kaolin clay were summarised in Table 3.3. The kaolin clay consists of 83% of silt and 17% of clay. The liquid and plastic limits were 61 and 38, respectively.

Table 3.3 Index properties of kaolin clay

Property	Value
Specific gravity, G_s	2.64
Liquid limit, LL (%)	61
Plastic limit	38
Plasticity index	23

3.9 Properties of Soil Specimens

The consolidation, permeability and shear strength properties of the foundation soil specimens were determined using two sets of soil samples. One set (Set 1) of soil samples were extracted from the foundation soil used in the unpiled model raft test.

The other set (Set 2) of soil samples were prepared in a consolidometer that was different from the one used for model tests. The properties of these two sets of soil samples were summarised in Table 3.4. The compression index (C_c), recompression index (C_r) and permeability (k_s) of the first set of soil samples were smaller than the second set of soil samples, whereas the undrained shear strength (c_u) is about 8% higher than the first set of soil samples. However, the effective friction angle was approximately the same for both sets of soil samples.

The settlement and pressure during the consolidation process of the clay slurry was monitored closely. The consolidation period was kept approximately constant. The square root time versus displacement curve and square root time is presented in Appendix A.

Each of the soil specimens used for model tests were characterised by undrained shear strength c_u obtained through in-situ probing (using portable vane shear apparatus), in-situ water content and c_u obtained from UU tests. After completion of the loading tests on model piled rafts, in-situ probing was carried out using portable vane shear apparatus at different locations as indicated in Figure 3.10 at two different depths, one at 50 mm below the surface and the other at 200 mm below the surface. The variation in c_u obtained at different locations within a specimen was small. The c_u obtained using portable vane shear apparatus was around 5 to 10% higher than the c_u obtained in UU tests. The UU test results are presented in Appendix B. The properties of different specimens are summarized in Table 3.5.

Table 3.4 Engineering properties of soil specimens

	Set 1	Set 2
Compression index (c_c)	0.43	0.48
Re-compression index (c_r)	0.14	0.15
Initial void ratio (e_o)	1.36	1.41
Water content (%)	46.0	50.0
Unit weight	16.3	16.4
Permeability (k_s), m/s	0.96×10^{-8}	0.98×10^{-8}
Undrained shear strength (c_u), kPa	40	37
c_u / σ_{vo}'	0.20	0.185
Undrained initial tangent modulus, kPa	8500	7500
Effective friction angle (°)	25	25

Figure 3.10 Locations for sample collection and in-situ probing

Table 3.5 Properties of soil specimens retrieved after model tests

Model test description	In-situ probing c_u (kPa)	water content (%)	UU-tests	
			c_u (kPa)	initial tangent modulus (E_u) (kPa)
UR40	44.0	46.2	40.0	8700
UR50	53.0	44.4	49.5	9600
B2L2	43.0	47.3	40.5	8800
B3L2	43.0	47.3	40.5	8800
B2L4	43.0	47.3	40.5	8800
B3L4	43.0	47.3	40.5	8800
4PB2L2	42.0	47.1	39.5	7800
4PB3L2	41.0	46.8	39.0	7500
4PB2L4	43.0	45.7	40.0	8000
4PB3L4	45.0	44.6	42.0	8800
9PB2L2	41.0	47.8	38.5	8000
9PB3L2	41.0	47.2	41.0	8600
9PB2L4	43.0	46.0	40.5	7600
9PB3L4	43.0	45.6	40.0	8400
16PB3L2	45.0	45.3	39.0	7800

3.10 Summary

The main objective of this research was to investigate the load settlement behaviour of rafts supported on piles with the view of gaining an understanding of how piles modify the settlement response of the corresponding unpiled raft. The nine model piled raft tests were configured with different number of piles with varying pile width and pile length. It was intended that the test results would help to unravel the effects of pile number, pile width and pile length on piled raft settlement. In addition, the measurement of interface contact pressure and axial load distribution in the piles would contribute towards a better understanding of the load sharing mechanism in a piled raft. The concurrent measurement of raft displacement and the mobilisation of interface contact pressure would provide a check on the validity of using Winkler springs for modelling the soil response beneath the raft.

Chapter 4

Experimental Results on Unpiled Raft and Single Piles

4.1. Introduction

In this chapter, the test results of the unpiled raft and isolated single piles are presented. Two preliminary unpiled raft tests were performed on model grounds of undrained shear strengths of 40 kPa and 50 kPa. These tests were intended to determine the appropriate model ground for the piled raft tests. One principal requirement in the selection of the model ground was that it should result in measurable raft settlement under the imposed loading.

After the model ground had been decided, load tests on model piles were conducted to determine the load-settlement characteristics and the ultimate capacity of each model pile when load tested in isolation. The test results from the unpiled raft and single piles were subsequently used as the reference benchmarks for analysing the behaviour of the piled raft.

4.2. Unpiled Raft

For unpiled rafts, loading was applied as a point load at the centre of the raft. The test was displacement-controlled at a constant displacement rate of 0.05 mm/min. The applied load (P), the displacements at the centre, corners and quarter points of the raft, and the raft-soil contact pressure at quarter and corner points were recorded at a frequency of 1 reading per minute. In this section, the load-settlement behaviour of the unpiled raft in both the tests is presented.

4.2.1. Load-settlement behaviour of an unpiled raft in clay with $c_u = 40$ kPa (UR40)

In order to maximise the information on the load-settlement characteristics of the unpiled raft, three complete load-unload cycles were carried out for each model test. Due to the length of the test, the three cycles were conducted over a period of 3 days. On the completion of unloading, the model raft was left in place but the data logger was allowed to continue to record the displacements of the raft. This would enable the relaxation and re-bounce to be monitored overnight. The results of the unpiled raft test conducted on a model ground with 40 kPa undrained shear strength would be described in detail in this section.

In the first cycle, the loading process took 4 hours and the unloading process was completed in 48 minutes. Thereafter, the model raft was left in place and data logging was continued. The second load cycle was conducted on the subsequent day, approximately 18 hours from the end of the first load cycle. In the second load cycle, the loading took 3 hours 16 minutes and the unloading was completed in 1 hour 6 minutes. The third load cycle was conducted on the 3rd day, about 16 hours from the end of second load cycle. Each load-unload cycle resulted in appreciable permanent settlement. The cumulative raft displacement attained at the loading process in the 3rd load cycle was about 50 mm or 0.167 times raft width (B_R).

Central settlement of raft under applied central point load (P)

In this study, the settlement at the centre of the raft would be denoted δ_c . The P - δ_c curve over the complete loading cycle is shown in Figures 4.1. It should be noted that there was a substantial time lapse between each loading cycle of not less than 12 hours.

Figure 4.1 shows that the P - δ_c response in the first load cycle was not linear even at relatively low imposed load. However, the non-linearity became increasingly more pronounced when the applied exceeded 11 kN. The initial gradient for the P - δ_c at zero load (k_o) was approximately 3.14 kN/mm. The raft was unloaded when the applied load reached 20 kN. At this load level, δ_c was 11.8 mm. During unloading,

as P was reduced from 20 kN to 14 kN, the LVDT reading showed that δ_c continued to increase by another 0.4 mm. The maximum δ_c in the first load cycle was 12.2 mm. As the applied load was reduced below 14 kN, δ_c decreased linearly with P . At the end of the unloading process, 1.9 mm of δ_c was recovered. After the load was completely removed, the LVDT continued to record an upward movement for another 3 hours. In total, about 2 mm of rebound was recorded within these 3 hours. The total permanent deformation measured in the first load cycle was 8.2 mm.

In the second load cycle, the P - δ_c response showed a more distinctive linear response during the reloading stage. There were no appreciable changes in the gradient of the slope as the load was increased up to 16 kN. Thereafter, the rate of change of δ_c with P increased gradually. Compared to the first load cycle, the gradient of the reloading path had increased quite appreciably. The slope was 4.14 kN/mm or 1.32 times the corresponding value in the first load cycle.

Figure 4.1 Applied central point load (P) versus central settlement (δ_c) of an unpiled raft in clay with $c_u = 40$ kPa

The unloading of the second load cycle began at 11.3 mm, or a cumulative δ_c of 20.1 mm for the 2 cycles. The P at the start of the unloading was 24.6 kN. During

unloading from 24.6 kN to 18 kN, δ_c continued to increase by a further 1 mm. Thereafter, δ_c reduced linearly with decrease in P . About 2.8 mm of δ_c was regained. Similar to the first load cycle after the complete unloading the LVDT at the centre of the raft recorded an upward movement for about 2 hour 20 minutes. Within this time, about 1.5 mm upward movement was recorded.

The P - δ_c behaviour in the third load cycle showed a linear response up to 20 kN. Beyond 20 kN, the curve tapered off very rapidly with increased δ_c . The loading cycle was terminated at a total displacement of about 50 mm ($0.16B_R$). At a total displacement greater than 30 mm ($0.1B_R$) the rate of increase of P with δ_c was about 20% of the k_o in the first load cycle.

Changes in loading path in different load cycles

In Figure 4.2, the δ_c in the second and third load cycles have been reinitialised to zero at the start of loading to compare the changes in response in the three load cycles. Considerable increase in k_o was observed in the second and third load cycles when compared to the first.

Figure 4.2 Applied central point load (P) versus central settlement (δ_c) of an unpiled raft in clay with $c_u = 40$ kPa – in different load cycles

The deformation and stiffness of the P - δ_c response are summarised in Table 4.1. The k_o in the reloading curves in the second and third load cycles was higher compared to the k_o in the virgin loading stage. The k_o in the second and third load cycles was about 1.32 and 1.38 times the k_o in the first load cycle.

Table 4.1 Deformation and stiffness of load (P) - central settlement (δ_c) response of an unpiled raft in clay with $c_u = 40$ kPa

	Cycle 1	Cycle 2	Cycle 3
Initial gradient at zero load (k_o) (kN/mm)	3.14	4.14	4.33
Total deformation (mm)	12.2	12.3	30
Recovered deformation (mm)	3.9	4.3	-
Permanent deformation (mm)	8.3	8.0	-
Loading period (minutes)	240	196	530
Unloading period (minutes)	48	60	-
Time elapsed before the start of next cycle (hours)	18	16	-

Settlement at corner and quarter points

The displacements at 3 corners of the raft were monitored during the loading test. The 3 corners were labelled 1 to 3, and the P - δ curves at these locations are presented in Figure 4.3.

In the first load cycle, the P - δ curves at all 3 corners were largely the same, with location 3 showing a slightly smaller settlement. At the end of the first loading stage, the δ at points 1, 2 and 3 was respectively 9.5 mm, 9.4 mm and 8.9 mm. The difference in δ at points 1 and 3, which were diagonally opposite to each other, was about 0.6 mm. By the end of the first loading stage the raft tilted by around 1.3° .

During unloading, from 20 kN to 14 kN, the LVDTs at all corner points recorded an additional settlement of 0.6 mm. Immediately upon complete unloading, the δ regained at all corner points was about 0.5 mm. Even after the load had been completely removed, the LVDTs at all corners continued to rebound for another 3 hours. The upward movement recorded was about 1.9 mm.

The k_o in the second load cycle was about 1.51 times the corresponding value in the first load cycle. At the end of the loading in the second loading cycle, the displacement at point 1, 2 and 3 was respectively 16.6 mm, 16.4 mm and 15.4 mm. The differential settlement between points 1 and 3 was about 1.2 mm. The tilting of the specimen was about 2.5° . No further tilting of the specimen was observed during unloading.

The displacements at 2 quarter points labelled 5 and 6 are as shown in Figure 4.4. The P - δ curves at points 5 and 6 were approximately the same. The deformation and stiffness of load-settlement response curves at corner and quarter points are summarised in Table 4.2. In the first load cycle, k_o at corner (average of three corner points) and quarter points (average of two quarter points) was approximately 1.7 times and 1.4 times the value at the centre.

Figure 4.3 Applied central point load (P) versus settlement at corner points of an unpiled raft in clay with $c_u = 40$ kPa

Figure 4.4 Applied central point load (P) versus settlement at quarter points of an unpiled raft in clay with $c_u = 40$ kPa

Table 4.2 Deformation and stiffness of load-settlement response at corner and quarter points of an unpiled raft in clay with $c_u = 40$ kPa

	Corner			Quarter		
Loading cycle	1	2	3	1	2	3
Initial gradient at zero load (k_o) (kN/mm)	5.4	8.2	8.2	4.33	5.7	5.7
Total settlement (mm)	9.3	9.9	27.9	10.5	11.0	26.1
Elastic deformation (mm)	2.3	2.4	-	2.5	2.8	-
Permanent deformation (mm)	7.0	7.5	-	8.0	8.2	-

Average settlement

The average settlement (δ_{avg}) of the raft was determined by fitting a curved surface to the measured settlements at the various points. The average settlement was determined by equating the volume created by the indentation of this surface with that generated by a horizontal plane. The P versus δ_{avg} response is shown in Figure 4.5. Figure 4.6 shows the same results but with the δ_{avg} in the second and third load cycles initialised to zero so as to provide a clearer comparison between the P versus δ_{avg} behaviour in different load cycles. The bearing capacity of the raft was interpreted based on the curve shown in Figure 4.5 and will be discussed in Section 4.2.3. The average settlement and the gradient of the loading curves are presented in Table 4.3. As to be expected the k_o of the loading path in the second and third load cycles was slightly higher than that in the first load cycle. The k_o in the second and third load cycles was very similar. Work hardening effect from the second and third load cycles was evident in the higher ground reaction observed in the third load cycle at comparable settlement after the onset of non-linear response.

Figure 4.5 Applied central point load (P) versus average settlement (δ_{avg}) of an unpiled raft in clay with $c_u = 40$ kPa

Figure 4.6 Applied central point load (P) versus average settlement (δ_{avg}) of an unpiled raft in clay with $c_u = 40$ kPa - in different load cycles

Table 4.3 Deformation and stiffness of load - average settlement response of a unpiled raft in clay with $c_u = 40$ kPa

Loading cycle	Average settlement, δ_{avg}		
	1	2	3
Initial gradient at zero load, k_o (kN/mm)	4.2	6.2	6.1
Total settlement (mm)	10.7	10.7	28.7
Recovered deformation (mm)	2.9	3.1	-
Permanent deformation (mm)	7.8	7.6	-

Differential settlement

Based on the displacement at various instrumented points on the raft at any applied load, it was clear that the deformed shape of the raft was in the form of a dish. At a given load, the displacement was highest at the centre and smallest at the corner. Hence, the difference between δ_c and δ at point 3 was taken as representative of the

differential settlement ($\Delta\delta$) of the raft. The changes in $\Delta\delta$ with applied load are shown in Figure 4.7. The test results show that $\Delta\delta$ increased approximately linearly with P up to $P = 20$ kN. The gradient of $\Delta\delta$ from zero to 20 kN was 8.6 kN/mm. During unloading, there was also a linear relationship between $\Delta\delta$ and P except at the start of the unloading process. The value of $\Delta\delta$ at $\delta_{\text{avg}} = 0.1B_R$ was about 3.8 mm.

The data shown in Figure 4.7 is re-plotted in Figure 4.8 by re-initialising the $\Delta\delta$ to zero at the beginning of the loading process in cycles 2 & 3. The rate of change of $\Delta\delta$ with P in all three load cycles was similar up to a load of 15 kN. Beyond this point, $\Delta\delta$ in second load cycle was slightly higher than the value obtained in first and third load cycles.

Figure 4.7 Applied central point load (P) versus differential settlement ($\Delta\delta$) of an unpiled raft in clay with $c_u = 40$ kPa

Figure 4.8 Applied central point load (P) versus differential settlement ($\Delta\delta$) of an unpiled raft in clay with $c_u = 40$ kPa - in different load cycles

4.2.2. Load-settlement behaviour of an unpiled raft in clay with $c_u = 50$ kPa (UR50)

A second test was carried out with the unpiled raft in clay with $c_u = 50$ kPa. Three load-unload cycles were applied. The details of each loading cycle is summarised in Table 4.4. The observation on the settlement response of the raft is discussed in the following sections.

Central settlement

Figure 4.9 shows the development of central settlement of the raft. The k_o in this case was approximately 6.1 kN/mm, this was about 1.9 times the value for the test done in foundation soil having an undrained shear strength $c_u = 40$ kPa. The unloading behaviour showed greater non-linearity. The k_o in the second and third load cycles was approximately 1.32 and 1.15 times the value obtained in the first load cycle. The deformation and stiffness of P - δ curves is summarised in Table 4.4

Table 4.4 Deformation and stiffness of load-settlement response curves on soil with $c_u = 50$ kPa

	Centre			Quarter			Corner			Average		
	1	2	3	1	2	3	1	2	3	1	2	3
Load cycle												
Initial gradient at zero load, k_o (kN/mm)	6.1	5.5	5.0	7.1	7.8	8.1	12.3	14.4	16.5	9.3	9.4	9.5
Total settlement (mm)	13.9	12.9	30	12.7	10.8	27.0	11.6	9.3	26.4	12.5	10.8	27.7
Recovered deformation (mm)	5.1	7.9	-	4.2	5.0	-	2.5	2.9	-	3.6	4.9	-
Permanent deformation (mm)	8.8	5.0	-	8.5	5.8	-	9.1	6.4	-	9.0	5.9	-
Loading period (minuets)	262	232		262	232	-	262	232	-	-	-	-
Unloading period (minuets)	113	126	-	113	126	-	113	126	-	-	-	-
Elapsed time before the start of next cycle (hours)	18	19	-	18	19	-	18	19	-	-	-	-

Figure 4.9 Applied central point load (P) versus central settlement (δ_c) of an unpiled raft in clay with $c_u = 50$ kPa

Figure 4.10 Applied central point load (P) versus settlement (δ) at corners of an unpiled raft in clay with $c_u = 50$ kPa

Figure 4.11 Applied central point load (P) versus settlement (δ) at quarter points of an unpiled raft in clay with $c_u = 50$ kPa

Figure 4.12 Applied central point load (P) versus average settlement (δ_{avg}) of an unpiled raft in clay with $c_u = 50$ kPa

Settlement at corner and quarter points

The settlements at the corner points 1, 2 and 3 are shown in Figure 4.10. The settlements at points 1, 2 and 3 were similar. No tilting of the raft was observed during the entire loading-reloading process. In the first load cycle, the average k_o of the points 1, 2 and 3 was about 2.2 times stiffer than that measured for the raft founded on a soil with $c_u = 40$ kPa (UR40). The average k_o in the second and third load cycles was about 1.17 and 1.34 times higher than the value obtained in the first load cycle. The settlements at quarter points 5 and 6 are shown in Figure 4.11. The LVDTs at the two locations essentially showed the same readings. The average settlements (δ_{avg}) determined by equivalent volume approach is shown Figure 4.12. The deformation and stiffness response were extracted and summarised in Table 4.4.

Differential settlement

The differential settlement was computed as the difference between the settlements at the centre and at the corner of the raft. The curve is shown in Figure 4.13. In the first load cycle, $\Delta\delta$ increased approximately linearly with P up to 20 kN. During unloading, no major change in $\Delta\delta$ was observed until P had reduced below 20 kN load. Thereafter, $\Delta\delta$ decreased nearly linearly with P . Unlike the test involving a foundation soil with $c_u = 40$ kPa (UR40), immediately upon complete unloading the settlement at the corner was greater than the settlement at the centre, resulting in negative value for the differential settlement $\Delta\delta$.

In the second cycle $\Delta\delta$ increased steadily upon loading up to 30 kN. Beyond 30 kN, only small changes in $\Delta\delta$ was observed. Upon complete unloading δ at the corner was again greater than δ_c , resulting in negative $\Delta\delta$. In the third load cycle it was observed that $\Delta\delta$ settled to a final steady value when the applied load exceeded 33 kN.

Figure 4.13 Applied central point load (P) versus differential settlement ($\Delta\delta$) of an unpiled raft in clay with $c_u = 50$ kPa

4.2.3. Bearing capacity of the unpiled raft (P_u)

For the two unpiled raft tests in two uniform soils with different shear strengths, the P - δ curves for the re-loading cycles exhibited typical response where the “yielding” was marked by the maximum load applied in the preceding load cycle. From the responses in the three load cycles, an envelope could be constructed (Figure 4.5 & 4.12). This envelope could be taken to be representative of a single stage loading response from 0 to 50 mm.

In the case of UR40 as seen in Figure 4.14, no definitive peaks were observed in the P - δ_{avg} curves. Even when the settlement had exceeded $0.1B_R$, P continued to increase with δ_{avg} . At a settlement of $0.15B_R$, the gradient of the P - δ_{avg} curve was about 7% of k_o in the first load cycle.

Figure 4.14 Applied central point load (P) versus average settlement δ_{avg} of an unpiled raft in clay with $c_u = 40$ kPa from model test and representative single stage response

Figure 4.15 Applied central point load (P) versus average settlement δ_{avg} of an unpiled raft in clay with $c_u = 50$ kPa from model test and representative single stage response

Evaluating the ultimate bearing capacity P_u from the P - δ_{avg} curves of this type (Figure 4.14 and 4.15) is fraught with uncertainties. The following are some methods available in the literature for evaluating P_u from the P - δ_{avg} curve.

1. **Log-log method:** In this method both the load and settlement are plotted on a log-log scale. The load corresponding to a distinctive marked change in the slope of the curve is taken as the ultimate load (DeBeer, 1970).
2. **$0.1B_R$ method:** In this method the ultimate load (P_u) is taken as the load corresponding to a limiting settlement equal to $0.1B_R$ (Briaud and Jeanjean, 1994).
3. **Tangent intersection method:** The ultimate load (P_u) is taken as the load corresponding to a distinctive marked change in the δ_{avg} (Trautmann & Kulhawy, 1988)

The P_u interpreted from the P - δ_{avg} curves based on the various suggested methods stated above is summarised in Table 4.5. The interpretation of P_u by the three methods is presented in Figure A-1 to A-6 of Appendix-A for both UR40 and UR50. For UR40, P_u determined using log-log method and tangent intersection method was about 0.37 times and 0.70 times the value obtained based on $0.1B_R$ method, respectively.

Table 4.5 Deduced bearing capacity of the unpiled raft by different methods

Method	P_u (kN)	
	UR40 ($c_u = 40\text{kPa}$)	UR50 ($c_u = 50\text{kPa}$)
Log-log method	11.0	18.0
Tangent intersection method	21.0	31.0
$0.1B_R$ method	30.0	36.0

Theoretical Bearing Capacity (P_u)

In this study, except for the small protrusion caused by the total stress transducers on the underside of the raft to measure the raft-soil pressure, no other surface features or special treatment were given to the well finished metal surface. The raft surface could be considered largely smooth. The undrained bearing capacity (P_u) of a surface raft is given by Eqn 4.1 (Terzaghi, 1943).

$$P_u = N_c s_c c_u A_R \quad \text{Eqn (4.1)}$$

where N_c is the bearing capacity factor, s_c is the shape factor, c_u is the representative undrained shear strength and A_R is the area of the raft. Since Terzaghi (1943), many modifications had been proposed by various authors for both the bearing capacity factor N_c and shape factor s_c . The P_u estimated based on these different proposed factors are presented in Table 4.6.

For $c_u = 40$ kPa and assuming a perfectly smooth raft, P_u was calculated to vary between 20.7 kN and 22.8 kN. When the raft was assumed rough, P_u was calculated to vary between 21.3 kN and 30.2 kN.

The P_u deduced from the experimental P - δ_{avg} curve based on the three methods ranged between 16 kN and 30.0 kN. Generally, it was found that the calculated theoretical P_u was smaller than the values obtained using the $0.1B_R$ method and were higher than the values obtained using the tangent intersection method and the log-log method.

Among the P_u values interpreted from the load tests, the value obtained by $0.1B_R$ method may be appropriate for square rafts [Cooke, 1986; Borel, 2001, Conte *et al.*, 2003; Cerato, 2005; and de Sanctis & Mandolin, 2006]. The $0.1B_R$ method may be even more appropriate in the present context when the rafts were to be supported on piles. Hence, P_u deduced from $0.1B_R$ method was adopted. The value of P_u was found to be 30 kN for UR40 and 36 kN for UR50.

Table 4.6 Estimated bearing capacity of the unpiled raft by different methods

Reference	Analysis type	Footing type	N_c	s_c	P_u (kN) UR40	P_u (kN) UR50
Terzaghi (1943)	Limit equilibrium	Rough	5.70	1.3	26.7	33.3
Meyerhof (1951)	Limit equilibrium	Rough	5.10	1.2	22.0	27.5
Hansen (1961)	Limit equilibrium	Rough	5.10	1.2	22.0	27.5
Skempton (1951)	Empirical	-	6.17	1.2	26.7	33.3
Shield and Drucker (1953)	Upper bound	Smooth	5.71	1.11	22.8	28.5
Michalowski and Dawson (2002)	Finite difference	Smooth	5.43	1.06	20.7	25.9
Michalowski (2001)	Upper bound	Rough	6.56	1.28	30.2	37.8
Salgado et al. (2004)	Numerical limit analysis (lower bound)	Rough	5.52	1.07	21.3	26.6
	Numerical limit analysis (upper bound)	Rough	6.22	1.21	27.1	33.9
Gourvenec et al. (2006)	Finite element analysis	Smooth	5.56	1.08	21.6	27.0
		Rough	5.91	1.15	24.5	30.6
	Upper bound	Rough	6.41	1.25	28.8	36.1

4.2.4. Interface Contact Pressure Distribution

It is well known that the bearing pressure at the base of the footing is not uniform. Although the average bearing pressure P/A_R is a convenient average value, it does not accurately represent the actual distribution of bearing pressure at the raft-soil interface. In this study the contact pressure (σ_c) variation across the raft-soil interface was measured by total pressure cells installed at the corner and quarter points. For convenience, a reference average applied pressure denoted as p_a was defined by dividing the applied load by the raft area (P/A_R).

Figure 4.16 shows the variation of σ_c at the 3 corner points with p_a in the first load cycle. The readings recorded by the total pressure cells at the three corner points

showed a high degree of agreement. σ_c increased at a relatively constant rate with applied load up to p_a of 165 kPa. Beyond this point, σ_c appeared to increase more rapidly. For $0 < p_a < 165$ kPa, the σ_c/p_a ratio was about 0.8. During unloading, σ_c initially showed only a small decrease as p_a was reduced. For most of the unloading process, σ_c/p_a was found to be greater than 1.

Figure 4.16 Applied central point load (P) versus interface contact pressure (σ_c) at corner points in first load cycle

Figure 4.17 Applied central point load (P) versus interface contact pressure (σ_c) at quarter points in first load cycle

Significantly, the contact pressure measured by the total pressure cells at the 2 quarter points showed interesting departures (Figure 4.17). Initially, the rate of increase in σ_c was slightly greater than unity up to $p_a = 80$ kPa. For $p_a > 80$ kPa, the rate of increase in σ_c reduced to about 0.75. Generally, it was found that σ_c at the corners of the raft remained higher than σ_c at quarter points. The differences became even greater during unloading stage.

Figure 4.18 shows the average response of the interface contact pressures at the 3 corner points and at the 2 quarter points in the first load cycle. From the plot, it is clear that during loading for p_a up to 110 kPa, the average σ_c at quarter points was slightly higher than the average σ_c at the corner points. For p_a beyond 110 kPa, the average σ_c at the corner points was higher than the average σ_c at quarter points and the difference between the two became bigger with increase in applied load. During the unloading, the average σ_c at corner points is substantially higher than the average σ_c at quarter points.

Figure 4.18 Applied central point load (P) versus average interface contact pressure at corner and quarter points in first load cycle

The p_a versus σ_c response in the second and third load cycle is presented in Figure A-7 to A-10 of Appendix A.

The variation of σ_c with raft displacements (δ) in the first load cycle is presented in Figure 4.19. The raft displacement was recorded by LVDTs located on the top face of the raft directly above the total stress transducer locations at points 1, 2, 3, 5 and 6. The development of σ_c with δ at the 3 corner points showed a high degree of agreement. Similarly, σ_c versus displacement curves at the 2 quarter points were also largely similar. At the initial stages, the curves for the quarter and corner points were hardly distinguishable. However, as the load increased further, σ_c at the corners began to register higher value. The difference continued to widen with increasing δ . During unloading, σ_c at the corners decreased rapidly with a steep gradient. At the quarter points, the drop in contact stresses was accompanied by further increase in displacement, which reversed direction only when σ_c fell below 100 kPa.

Figure 4.19 Measured interface contact pressure (σ_c) versus raft settlement (δ) response in first load cycle

The σ_c versus δ response in the second and third load cycle and for the entire loading process at corner and quarter points is shown in Figure A-7 and A-8 of Appendix - A. The average applied pressure (p_a) versus δ_{avg} is also presented in these figures. In the first and second load cycles, at any given displacement, the σ_c at corner and quarter points was higher than the p_a .

In Winkler's hypothesis, the development of contact stresses with raft displacement was modelled by linear soil springs with constant spring stiffness. In practice the linearity assumption is only approximately true, even at low stress level. The variation of contact pressure with unpiled raft settlement would be discussed with the view of gaining a better understanding of how the non-linearity of the Winkler's spring could be approximated. Furthermore, if the stiffness of a linear Winkler's spring is required either for computational simplicity or as demanded by existing computer programs, it may be appropriate to find out how the constant linear spring value should be estimated.

The secant soil stiffness (k_s) could be one way of approximating the Winkler soil spring constant. The secant stiffness may be defined as the ratio of contact pressure divided by settlement (σ_c/δ). Due to the non-linearity of the $\sigma_c - \delta$ response, k_s will vary with load level. Hence, in addition to the initial k_s at very small load level, the

k_s at loads equal to $1/3P_u$, $1/2P_u$, $2/3P_u$ (secant stiffness) was deduced. The k_s values obtained from different load cycles of the model tests are presented in Table 4.7.

Furthermore, a simplified measure of the stiffness of the raft-contact pressure response (k^*) could be defined as the ratio of applied pressure to the average raft settlement (p_d/δ_{avg}). The k^* at various load levels corresponding to $1/3P_u$, $1/2P_u$, $2/3P_u$ are presented in Table 4.7. The variation of secant soil stiffness along the diagonal of the raft is presented in Figure 4.20. The k_s at corner points, at $1/3P_u$, $1/2P_u$ and $2/3P_u$ was approximately 0.93, 0.89 and 0.91 times the k^* at corresponding loads, respectively. Similarly, the k_s at quarter points for load levels of $1/3P_u$, $1/2P_u$ and $2/3P_u$ was found to be approximately 0.73, 0.63 and 0.64 times k^* at the corresponding load levels, respectively.

Figure 4.20 Variation of soil spring stiffness along diagonal of the raft

Table 4.7 Soil stiffness, k_s (kPa/mm) at corner and quarter points – UR40

Load cycle	Load range (kN)	Corner points				Quarter points			Average at corner points	Average at quarter points	average stiffness (k^*)
		1	2	3		5	6				
Cycle 1	Initial stiffness	60.0	60.0	60.0		60.0	60.0		60.0	60.0	46.7
	0 – 10	45.2	42.1	42.4		33.2	34.5		43.2	33.9	46.7
	0 – 15	30.1	27.2	28.2		20.4	21.1		28.5	20.7	32.2
	0 – 20	21.1	19.6	19.8		14.5	13.8		20.2	14.2	22.2
Cycle 2	Initial stiffness	79.1	76.7	74.2		48.0	51.0		76.7	49.5	68.9
	0 – 10	79.1	76.7	74.2		48.0	51.0		76.7	49.5	68.9
	0 – 15	69.1	67.3	64.8		40.9	43.0		67.1	42.0	68.9
	0 – 20	54.5	50.5	51.1		30.5	31.9		52.1	31.2	52.9
Cycle 3	Initial stiffness	124.0	119.1	116.3		85.0	91.2		119.8	88.2	67.8
	0 – 10	124.0	119.1	116.3		85.2	91.2		119.8	88.2	67.8
	0 – 15	106.2	100.1	99.5		73.9	78.5		101.9	76.2	67.8
	0 – 20	86.6	78.8	81.3		61.1	64.0		82.2	62.5	67.8

Figure 4.21 Applied central point load (P) versus average settlement (δ_{avg}) for UR40 and UR50

Choice of Foundation Soil

Figure 4.21 shows the P versus δ_{avg} response for UR40 and UR50. As to be expected at a given applied load, the δ_{avg} was much higher in the softer foundation soil (UR40) except at the initial stages when P was small. For the purpose of the present study, a larger settlement was a desirable trait for the unpiled raft as the larger settlement would help to accentuate the settlement reducing effect of piles in raft. If the firmer soil was chosen, the smaller settlement that ensured, particularly when piles are included, would give rise to greater uncertainties in measurement accuracy.

4.3. Load Tests on Single Piles

To investigate the effect of piles on the settlement behaviour of raft, four different model square piles with different widths and different lengths were used. It was important to know the characteristics of each of these piles. In particular, the load bearing capacity, the load-settlement response and the contribution from shaft and end bearing resistance and its rates of mobilisation are important parameters. In this

section the test results on isolated single piles are presented. The load tests were displacement-controlled with a penetration rate of 0.05mm/min; this was the same rate used in the unpiled raft tests. For ease of reference, the experiments on single piles were designated systematically based on the pile width and pile length. Thus, pile with a breadth (B) of 30 mm and length (L) of 400 mm was designated as B3L4 and piles with a breadth of 20 mm and length of 200 mm would be designated as B2L2. To check the repeatability of the test, the load test on pile B3L4 was repeated in the same foundation soil.

The load versus settlement curve for the single pile tests are shown in Figure 4.22 and Figure 4.23. These tests were carried out on the softer foundation soil with $c_u=40$ kPa. A well-defined failure load was observed for all the piles. The ultimate load (Q_u) mobilised by the piles, the displacement at which ultimate load was mobilised and the average slope of pre-failure load-displacement curve (k_p) are extracted and summarised in Table 4.8. Table 4.8 also shows the load-settlement ratios calculated using the expression provided by Fleming *et al.* (1992) for load-settlement ratios for piles in a homogeneous elastic medium. The pile stiffness deduced from the experimental load-settlement curves was about 20% to 40% smaller than those calculated using the expression given by Fleming *et al.* (1992).

Figure 4.22 shows the load-settlement response of pile B3L4 obtained in two load tests conducted in the same foundation soil. The ultimate loads obtained in the two tests were 1.0 and 1.1 kN, respectively. The initial tangent stiffness for both piles was practically the same.

Table 4.8 Ultimate loads mobilised by different piles

Pile	Ultimate load, Q_u (kN)	Displacement at ultimate load (mm)	Pile stiffness, k_p (kN/mm)	Fleming <i>et al.</i> (1992) k_p (kN/mm)
B2L2	0.41	0.40	1.03	1.43
B2L4	0.68	0.52	1.31	2.21
B3L2	0.62	0.45	1.38	1.75
B3L4	1.10	0.55	2.00	2.56

Figure 4.22 Pile head load versus pile head settlement for single isolated pile B3L4 ($B = 30$ mm & $L = 400$ mm)

Figure 4.23 Pile head load versus pile head settlement for piles B2L2, B3L2 and B2L4

The piles in the load tests were instrumented with strain gauges at three locations as described in Chapter 3. The top strain gauge was placed 2 cm below the pile head and could provide an estimate of the pile head load. Another strain gauge was

placed 2 cm from the pile tip. The third was located at the mid-level of the pile. The axial load was calculated by multiplying the measured strains with the cross sectional area and Young's modulus of the pile. The shaft resistance was extrapolated from the strain gauge readings at the pile head and near the pile tip using the following expression.

$$Q_s = (Q_1 - Q_4) \frac{L}{L - 20} \quad \text{Eqn (4.2)}$$

Where:

Q_s = shaft resistance

Q_1 = axial load at pile head

Q_4 = axial load at 20 mm above pile tip, L = length of the pile

The end bearing resistance (Q_b) was determined using the following expression,

$$Q_b = (Q_1 - Q_s) \quad \text{Eqn (4.3)}$$

Figures 4.24 to 4.27 show the mobilisation of shaft and end bearing resistances with pile head displacement. In all cases, the ultimate shaft resistance was fully mobilised at a pile head displacement of about 0.5 mm.

Figure 4.24 Variation of end bearing and shaft resistance with pile head settlement for pile B2L2 ($B = 20$ mm, $L = 200$ mm)

Figure 4.25 Variation of end bearing and shaft resistance with pile head settlement for pile B2L4 ($B = 20$ mm, $L = 400$ mm)

Figure 4.26 Variation of end bearing and shaft friction resistance with pile head settlement for pile B3L2 ($B = 30$ mm, $L = 200$ mm)

Figure 4.27 Variation of end bearing and shaft friction resistance with pile head settlement for pile B3L4 ($B = 30$ mm, $L = 400$ mm)

The end bearing capacity can be estimated by the following empirical equation

$$Q_b = q_b A_b = N_c c_u A_b \quad \text{Eqn (4.4)}$$

Where:

Q_b = end bearing capacity

N_c = bearing capacity factor

c_u = undrained shear strength at pile base

A_b = area of the pile base

The mobilised ultimate shaft and end bearing resistances from the single pile load tests are presented Table 4.9. By comparing the empirical ultimate end bearing resistance calculated using known c_u values in Eqn 4.4 with the end bearing resistance obtained in the tests, the bearing capacity factor N_c was back-calculated. For piles B2L2 and B3L2 and B3L4, the back-calculated N_c factor was 6.9, 6.4 and 8.6, respectively. For pile B2L4 the back-calculated N_c factor was about 10.6. For all piles except B2L4, the back-calculated N_c factor was smaller than 9, a value which was commonly assumed in design calculations.

Table 4.9 Shaft and end bearing capacities of model piles

Pile	Ultimate end resistance (kN)	Back-calculated bearing capacity factor (N_c)	Ultimate shaft resistance (kN)	Back-calculated adhesion factor (α)
B2L2	0.11	6.9	0.32	0.50
B2L4	0.17	10.6	0.63	0.49
B3L2	0.23	6.4	0.46	0.48
B3L4	0.31	8.6	0.98	0.51

The shaft capacity of a pile in cohesive soil can be estimated using the following equation (Tomlinson, 1957)

$$Q_s = \alpha c_u A_s \quad \text{Eqn (4.5)}$$

Where:

α = adhesion factor

c_u = average undrained shear strength over the length of pile

A_s = surface area of the shaft

The undrained shear strength factor (α) was back-calculated by equating the mobilised ultimate shaft resistances with the theoretical values and the known c_u using Eqn 4.5. The α values back-calculated from the pile load tests varied from 0.48 to 0.51. This falls well within the typical range of α values that had been reported by other researches using field and laboratory test data (Vesic, 1977; Fleming *et al.*, 1992; Cuduto, 1999; Horikoshi, 1999).

4.4. Summary

Unpiled raft

In this chapter the tests on unpiled raft and single individual piles were discussed. Two tests were conducted on the unpiled raft on model ground with c_u of 40 kPa and 50 kPa to assess the load-settlement behaviour. The tests were performed to help in the decision concerning the appropriate model ground for the main test program. As expected, the average settlements were higher in the case of UR40 compared to UR50. The initial k_o in UR40 was 0.5 times the value in UR50. The model ground with c_u of 40 kPa was chosen for all subsequent tests.

In the two unpiled raft tests, the P - δ_{avg} curves for the three load-unload cycles showed a very well-defined classical response. The “yielding” in the succeeding cycles corresponded very closely to the maximum load applied in the preceding load cycle. An envelope was constructed for the load-settlement response over the 3 load-unload cycles. The ultimate bearing capacity of the raft was deduced from this envelope using various interpretation methods. The P_u deduced based on the $0.1B_R$

method was 30 kN and 36 kN for UR40 and UR50, respectively. The log-log method and the tangent intersection method yielded lower values for P_u . For UR40, P_u obtained using log-log method and tangent intersection method was about 11 kN and 21 kN, respectively.

The total pressure transducers at the corners and quarter points recorded very consistent results with no more than 12% difference between the various transducers. The p_a versus σ_c responses at three corner points were in good agreement. Similarly, the p_a versus σ_c response at two corner points showed very consistent response. At $1/2P_u$, the σ_c/p_a ratio at corner and quarter points was respectively 0.8 and 0.6.

The σ_c - δ curves at different location of the raft were highly non-linear. At small displacement (i.e. for $\delta < 0.5$ mm) the σ_c - δ curves at different locations were approximately the same. For $\delta > 1.5$ mm, σ_c mobilised at corner points was higher than the corresponding value at quarter points, and the difference was amplified with increasing δ .

Load Tests on Single Piles

In total 5 load tests were conducted on 4 model piles of different dimensions. The pile load tests showed very well-defined load-settlement response. The on-set of ultimate load was clearly marked. In all cases, the ultimate load was mobilised at pile head settlements of between 0.4 mm to 0.55 mm. A correlation of the test data with empirical pile equations show that the α factor varied between 0.48 to 0.51. The end-bearing capacity factor N_c showed a larger variation, ranging from 6.4 to 10.6. The lower N_c factors were associated with shorter piles.

Chapter 5

Load-Settlement Behaviour of Piled Rafts under Axial Load

5.1 Introduction

In this chapter the results obtained from 9 model piled raft tests would be presented. The tests comprise of four 4-pile piled rafts, four 9-pile piled rafts and one 16-pile piled raft. In each of the four 4-pile piled rafts and 9-pile piled raft, either the pile length or the pile width would be changed. All the model piled raft tests were displacement controlled with a rate of penetration of 0.05 mm/min, similar to the rate used in the unpiled raft and single pile tests. The model piled raft tests were conducted on a model ground of undrained shear strength of 40 kPa (± 2 kPa). The loading was done in 3 cycles. However, in the following sections attention will be focussed on the behaviour in the first loading cycle.

The load-settlement behaviour of 16-pile piled raft would be discussed in detail in the following section. In the subsequent sections, the effect of number of piles, length of the piles and width of the piles in modifying the piled raft settlements would be presented.

5.2 Test on Model 16-Pile Piled Raft (16PB3L2)

The 16-pile piled raft comprised piles of 30 mm width (B) and 200 mm length (L). The centre to centre spacing between the piles was $2.5B$. The c_u of the model ground was approximately 39 kPa. In the following section the load-settlement response and load transfer through piles and raft-soil interface for the model 16-pile piled raft is presented.

Central settlement, δ_c

Figure 5.1 shows the applied load (P) versus central settlement (δ_c) response for the 16-pile piled raft and unpiled raft. In the case of the 16-pile piled raft, the settlement increased nearly at a constant rate until P reached 9 kN. Beyond this point, there was an appreciable increase in the rate of settlement with P . Figure 5.1 also shows that at the same P , the 16-pile piled raft experienced a much smaller δ_c than the unpiled raft. The percentage reduction in δ_c gradually decreased as P increased.

Figure 5.1 Applied central point load (P) versus central settlement (δ_c) response of 16-pile piled raft (16PB3L2)

The settlement reduction due to the introduction of piles is presented in terms of percentage reduction of the unpiled raft settlement in the lower graph in Figure 5.1. It was observed that the maximum reduction of about 70% occurred at a load of 6 kN and reduced steadily as P increased. The final reduction was found to be slightly less than 50% when P reached 20 kN.

Settlement at corners

Figure 5.2 shows P versus δ response at the three corners of the raft. The P versus δ curves at the three corners were largely similar. It showed that the raft had settled almost symmetrically with no significant sign of tilting.

Based on the settlement obtained at instrumented locations on the raft, the deformed shape of the raft was of a dish, similar to that of the unpiled raft. At any given load, δ_c was highest at the centre and smallest at the corners. The difference between the central and corner settlement was taken as representative of the raft differential settlement ($\Delta\delta$).

Figure 5.2 Applied central point load (P) versus settlement (δ) at 3 corners of 16-pile piled raft (16PB3L2)

Differential settlement, $\Delta\delta$

Figure 5.3 shows the variation of $\Delta\delta$ with P both for unpiled raft and 16PB3L2. As the applied load increased from zero up to 9 kN, $\Delta\delta$ in the piled raft increased at a smaller rate of about 0.03 mm/kN when compared with the unpiled raft. Beyond 9 kN, the rate of increase in $\Delta\delta$ accelerated to about 0.09 mm/kN.

Figure 5.3 Applied central point load (P) versus differential settlement ($\Delta\delta$) of 16-pile piled raft (16PB3L2)

The differential settlement in the piled raft was considerably smaller when compared to the unpiled raft. The percentage reduction in differential settlement is highest at small load level and decreased as P increased. The percentage reduction in $\Delta\delta$ decreased rapidly as P increased from 6 kN to 13 kN, from about 80% to about 54%.

Distribution of applied load between raft and piles

The total applied load was resisted by both pile resistance and the bearing pressure beneath the raft. Due to the manner by which these soil resistances were mobilised, the load sharing between pile and raft was not a constant but changed with load level. The stiffer pile tends to carry a bigger share of the load at the initial stages when the settlement was small. After the piles had mobilised their full capacities, the subsequent increase in load was transferred as bearing pressure beneath the raft. With the stress cells and strain gauges installed in the raft and piles, it was possible to monitor the load distribution between piles and raft. In the following sections the percentage of applied load transferred through piles and through bearing pressure beneath raft would be examined.

Load transfer through piles

As highlighted in previous chapters, the piles in the model piled rafts were instrumented with strain gauges to measure the axial load at different elevations of the piles. In 16PB3L2, three piles were instrumented. The mobilisation of the geotechnical resistance of these piles is shown in Figure 5.4. The inset shows the location of the instrumented piles in the raft.

Figure 5.4 shows that, as the applied load increased from zero to about 5 kN the geotechnical resistance mobilised by the three piles was nearly identical. Beyond this, pile 1 located in the central core mobilised slightly higher resistance compared to the other two piles. Within this load of P , pile 3 mobilised the least resistance. Pile 1 mobilised its ultimate capacity (Q_u) slightly earlier than the other two piles; it mobilised its ultimate resistance when the applied load on raft reached 11 kN, whereas the ultimate capacities in the other two piles were fully mobilised when P

reached 13 kN. At full mobilisation, the difference between Q_u in the three piles was less than 5%. The Q_u of pile 1 and pile 2 was about 0.58 kN and that of pile 3 was 0.55 kN. It should also be noted that these values were about 5% and 9% smaller than the ultimate capacity of the pile (B3L2) tested as a stand-alone single pile (see Figure 4.22). However, since the tests were conducted on model ground prepared separately, the inevitable small variation in the undrained shear strengths of the model ground would have contributed partly to the observed differences in the mobilised pile resistance in the piled raft.

Figure 5.4 Mobilisation of geotechnical resistance of piles with applied load on raft in 16PB3L2

Load transferred through raft bearing pressure

In the case of 16PB3L2, the raft-soil interface contact pressure was measured at 5 locations distributed across the base of the raft; three at the corners and two at quarter points. Figure 5.5 shows the monitoring locations and the variation of σ_c with p_a at the three corner points. The mobilised σ_c at corner points 1 and 2 showed

a high degree of agreement. The σ_c at corner point 3 was slightly smaller. Initially σ_c was mobilised at a near constant rate up to $P = 13$ kN. It should be noted that $P = 13$ kN also corresponded roughly to the load level at which full mobilisation of pile capacities occurred. For P up to 13 kN or at the equivalent applied average pressure of $p_a = 145$ kPa, σ_c/p_a at corner points 1 and 2 was about 0.53 and σ_c/p_a at corner point 3 was about 0.45. The rate of increase of contact pressure appeared to undergo a transition for P between 14 kN to 18 kN. For P beyond 18 kN, σ_c was found to increase at the same rate of change of p_a , that is $d(\sigma_c)/d(p_a) = 1$. This implied that all subsequent increase in the applied load was transferred to the soil as interface contact pressure, apparently with no further contribution from the piles.

Figure 5.5 Mobilisation of interface contact pressure with applied load on raft at corners in 16PB3L2

Figure 5.6 Mobilisation of interface contact pressure with applied load on raft at quarter points in 16PB3L2

Figure 5.6 shows the variation of σ_c with p_a at the two quarter points labelled 5 & 6. The mobilised σ_c at quarter point 5 was slightly smaller than the σ_c at quarter point 6. As P increased from 0 to 18 kN, σ_c was mobilised at a near constant rate. The σ_c/p_a ratio at points 5 and 6 in the initial stages was about 0.40 and 0.45, respectively. For loads beyond 18 kN, the σ_c was mobilised at a somewhat higher rate. The ratio at both points was approximately the same for loads beyond 18 kN, having a magnitude of 0.70. Compared to the corners, it appeared that the pressure at the internal quarter points increased at a slower rate. A persistent differential in $d(\sigma_c)/d(p_a)$ ratio would lead to greater non-uniformity in interface contact pressure distribution across the raft-soil contact surface.

Figure 5.7 Reduction in interface contact pressure in 16PB3L2 compared to unpiled raft

In Figure 5.7, the interface contact pressure for the corner was the average value derived from the three readings at the corners. Similarly the contact pressure for the quarter point was the average of the readings from the two quarter points. At all load level, σ_c at quarter point was smaller than the σ_c at the corner. The rate of increase in σ_c at quarter points was also smaller than the rate of increase of σ_c at corners. Beyond $P = 16$ kN, a sudden change in the rate of increase of σ_c at the corner was observed. The gradient of the roughly straight section of the graph was nearly one (i.e. $d(\sigma_c)/d(p_a) = 1$). Similar trend was observed for the contact pressure at the quarter point, with $d(\sigma_c)/d(p_a)$ approximately equal to 0.7.

Figure 5.7 shows that the contact pressure in the piled raft was considerably smaller than that at corresponding locations in the unpiled raft. At quarter points, the reduction in σ_c gradually increased with P up to about 10 kN. For loads beyond this the reduction in σ_c at quarter points compared to the unpiled raft was nearly constant. At 10 kN and 15 kN, the reduction in σ_c at quarter points compared to unpiled raft was about 40 kPa.

At the corners of the raft, the reduction in σ_c progressively increased with P throughout the loading range. At 10 kN and 15 kN, the contact pressure was reduced by about 25 kPa and 35 kPa, respectively, compared to unpiled raft.

Figure 5.8 shows the load sharing between piles and raft. The average raft bearing pressure was estimated by assuming that the contact pressure measured at quarter points and corner points was effective over $2/3$ and $1/3$ of the area of the raft, respectively. The raft bearing resistance was obtained by multiplying the estimated average raft bearing pressure with the net raft contact area. The net raft contact area was calculated as the total raft area minus the area occupied by piles.

As the applied load on the raft, P , was increased from 0 to 10 kN, the piles initially carried about 55% of the applied load, gradually increased to a peak of about 80%. The balance of the applied load was transmitted as raft-soil interface contact pressure. The proportion of the load carried by piles decreased as P was increased beyond 10 kN. Figure 5.8(a) shows the curve representing the load carried by piles changed very little for $P > 10$ kN, showing that there was no further increase in contribution from the piles. As a consequence, all subsequent increase in P must be resisted by the raft-soil contact pressure. This is clearly seen in Figure 5.8(a) as a gradual pick up in gradient of the curve representing the proportion of load carried by the raft. Beyond 18 kN, it can be observed that all the increase in applied load was fully resisted by the increase in raft bearing pressure. This is evident from the agreement in the slopes of the graphs depicting total applied load and raft bearing resistance in Figure 5.8(a).

From the load test on isolated single piles, pile B3L2 was found to mobilise its ultimate resistance at a pile head displacement of about 0.5 mm. In contrast, the same pile when incorporated in a 16-pile piled raft was found to mobilise its ultimate resistance only when the pile head displacement reached 1.5 mm. The delayed mobilisation of the pile resistance could be attributed to complex soil-pile-raft interaction. Such a response would have important implication in the simulation of piled raft behaviour using the Winkler springs or subgrade reaction approaches in which no account was given to pile-soil-pile interactions.

Figure 5.8 Variation of load sharing between piles and raft with applied load in 16PB3L2

To check for load equilibrium, the sum of the raft and pile resistance was plotted against the applied load in Figure 5.8. The applied load and developed resistances were balanced quite reasonably for P up to about 6 kN. For P between 6 kN and 15 kN, the developed resistance was slightly higher than the applied load. For $P > 15$ kN, the sum of the raft and pile resistances was smaller than the applied load. The maximum difference was about -8% and occurred at $P = 20$ kN.

There are two possible explanations to the observed discrepancy. First and foremost was the uncertainty regarding the actual pressure distributions in the raft-soil interface. In the computation, an area weighted approach and pressure transducer

readings were adopted. This might not be fully representative of the actual situation as the interface contact pressure, and its distribution was found to vary with applied load level. The other possible source of discrepancy was the frictional resistance at the vertical sides of the raft.

5.3 Factors Affecting Settlement of Piled Raft

In this section, the effect of various pile parameters in modifying the central settlement (δ_c) of an unpiled raft would be discussed. The data were derived from the tests done on four 4-pile piled raft and four 9-pile piled raft in conjunction with the data derived from the 16-pile piled raft discussed in the preceding sections. The scope of the variables under investigation was necessarily restricted in view of the time-consuming nature of the experiment and the complexities involved. Hence, the focus would be given to the following parameters: number of piles, the length and width of piles. In addition, the effect of the load-settlement stiffness and the load bearing capacities of the piles would also be examined.

5.4.1 Effect of number of piles on piled raft settlement

The P versus δ_c response of 4PB3L2, 9PB3L2, 16PB3L2 and unpiled raft is shown in Figure 5.9(a). The percentage reduction expressed with respect to the unpiled raft settlement is shown in Figure 5.9(b). Figure 5.9(a) shows that the P versus δ_c responses of 4B3L2 and 9PB3L2 were only approximately linear at small load level, they became increasingly non-linear as the load increases.

As expected, the 4-pile piled raft shows the smallest amount of settlement reduction. However, the 9-pile and 16-pile piled rafts showed no major differences in the amount of reduction. In all three piled rafts, it was observed that the percentage reduction in δ_c show a peak at P approximately equal to 5 kN. For $P > 5$ kN, the percentage reduction in the 4-pile piled raft decreased very rapidly. At $P = 18$ kN, there was no discernable difference between the unpiled raft and the 4-pile piled raft. It should be noted that the piles in 4PB3L2 were 200 mm long, which was smaller than the width of the raft. At large raft displacement, the load-settlement behaviour of the piled raft was dominated by the bearing capacity of the

raft. The 200 mm long piles were located well within the failure zone of the raft, and hence had little influence on the load-settlement characteristics at high load level.

(a)

(b)

Figure 5.9 Applied central point load (P) versus central settlement (δ_c) of 4-, 9- and 16-pile piled rafts with piles of $B = 30$ mm and $L = 200$ mm

Nevertheless, in practical application the working load would be well below that required to trigger bearing capacity failure. Therefore, even with 4 B3L2 piles ($Q_u = 0.62$ kN), considerable reduction in δ_c was observed for P up to 15 kN in 4PB3L2. For this particular raft, the peak value of settlement reduction was 40% at $P = 5$ kN. At 10 kN, the percentage reduction in δ_c was about 24%.

Increasing the number of piles in the raft from 4 to 9 caused a major reduction in settlement. In the case of 9PB3L2, the maximum reduction in δ_c of close to 75% occurred at $P = 5$ kN; it then reduced progressively as P was further increased, but at a much slow rate compared to 4PB3L2. At $P = 10$ kN and $P = 15$ kN, the settlement reduction was 65% and 55%, respectively.

Figure 5.9 shows that increasing the number of piles to 16 nos did not result in further settlement reduction when compared to just having 9 piles. It was apparent that the load-settlement behaviour of 9PB3L2 and 16PB3L2 were significantly different from the unpiled raft, resulting in higher load bearing capacity for the piled rafts and smaller settlement. Clearly then, the failure mechanisms in the piled rafts were not similar to the unpiled raft. One possible explanation for the observed behaviour was that the bearing capacities of the piled rafts were governed by block failure mechanism, instead of individual pile failure. This suggested that for a given pile size, increasing the number of piles beyond certain limit might not result in further reducing settlement. Hence, the proper selection of pile configuration, number and pile spacing would be an important consideration in the deployment of piles for reducing raft settlement.

Figure 5.10 shows P - δ_c curves of 4PB2L2, 9PB2L2, 4PB2L4, 9PB2L4 and unpiled raft. The key difference between Figure 5.9 and Figure 5.10 was the length of pile used. In 4PB2L2 in which the piles had capacity $Q_u = 0.42$ kN, about 20% reduction in δ_c was achieved at $P = 5$ kN. At $P = 10$ kN, the settlement reduction was about 12%. Similar to 4PB3L2, the P - δ_c curve for 4PB2L2 converged to that of the unpiled raft when P was greater than 17.5 kN. This confirms that the failure mechanism was indeed dominated by bearing capacity failure of raft. The sparsely located short piles encompassed within the active wedge did not provide any additional capacity in the ultimate state.

Increasing the number of B2L2 piles from 4 to 9 increased the settlement reduction from 12% to 36% for $P = 10$ kN and from 10% to 40% at $P = 15$ kN. However, as can be seen from Figure 5.10(b), the gradient of the curve for 9PB2L2 was found to be quite steep even at large P . Increasing the number of B2L4 piles from 4 to 9

increased the settlement reduction from 35% to 55% for $P = 10$ kN and from 29% to 51% at $P = 15$ kN.

Figure 5.10 Applied central point load (P) versus central settlement (δ_c) of 4- and 9-pile piled rafts with piles of $B = 20$ mm, $L = 200$ mm & 400 mm

Figure 5.11 shows P - δ_c curves of 4PB3L4, 9PB3L4 and unpiled raft. Compared to Figure 5.9, the length has increased from 200 mm [B3L2] to 400 mm [B3L4]. From the load tests on single isolated piles, the ultimate load bearing capacity was 0.62 kN for B3L2 and 1.1 kN for B3L4. This translated into an increase of 77 % for each pile. Figure 5.11(b) shows that the settlement reduction against applied load curve for the 4-pile piled raft showed a maximum of 70%. As P increased from

0 to 8 kN, 4PB3L4 and 9PB3L4 showed essentially the same response. For $P > 8$ kN, 9PB3L4 began to show greater reduction in settlement. At $P = 10$ kN, increasing the number of piles from 4 to 9 increased the settlement reduction from 58% to 69%.

Figure 5.11 Applied central point load (P) versus central settlement (δ_c) of 4- and 9-pile piled rafts with piles of $B = 30$ mm and $L = 400$ mm

5.4.2 Effect of pile length

Figure 5.10 shows the P - δ_c curves of 4PB2L2, 9PB2L2, 4PB2L4, 9PB2L4 and unpiled raft. Changing the pile length from 200 mm (B2L2) to 400 mm (B2L4) resulted in major increase in settlement reduction. The load bearing capacity of the B2L2 and B2L4 determined from single pile load tests was 0.42 kN and 0.68 kN, respectively. The increase in pile length caused the settlement reduction to change from 20% to 50% in the 4-pile piled raft when $P = 5$ kN, and from 12% to 34% at $P = 10$ kN. In the 9-pile piled raft, the change in pile length caused the settlement reduction to change from 21% to 67% at $P = 5$ kN, and from 38% to 56% at $P = 10$ kN.

Figure 5.12 shows the P - δ_c response of 4PB3L2, 4PB3L4, 9PB3L2 and 9PB3L4 and unpiled raft. The main difference between Figure 5.10 and Figure 5.12 lies in the width of the pile. Figure 5.10 shows the tests results involving pile width of 20 mm while Figure 5.12 shows the results involving pile width of 30 mm. Figure 5.12 clearly shows that raft 9PB3L4 with a larger number of long pile had the largest bearing capacity, followed by raft 9PB3L2. For the 4-pile piled raft, increasing the pile length had also led to significant change in ultimate bearing capacity and settlement reduction. It was observed that for P less than 7 kN, there was no appreciable difference in the P - δ_c curves. The curves only began to diverge when P was greater than 10 kN.

From Figure 5.12(b), the maximum reduction in the unpiled raft settlement due to the addition of piles is about 40% for 4PB3L2 and about 70% for the other three rafts. The results also showed that settlement reduction for 9PB3L4 did not vary appreciably with P , hovering around 70% for P up to 20 kN. In contrast, the percentage reduction in settlement was observed to decrease as P increased for the other rafts. As noted earlier, for the 4-pile piled rafts, the load-settlement behaviour was dominated by bearing capacity failure mechanism of the raft; the effect of the short piles was totally obliterated at large P in 4PB3L2. This occurred when P was greater than about 17 kN.

Figure 5.12 Applied central point load (P) versus central settlement (δ_c) of 4- and 9-pile piled rafts with piles of the same B of 30 mm, & $L = 200$ mm and $L = 400$ mm

5.4.3 Effect of width of the piles

To understand how changes in pile width would affect the performance of the raft, the $P - \delta_c$ response of 4PB2L2, 4PB3L2, 9PB2L2, 9PB3L2 and unpiled raft are presented together in Figure 5.13. The pile length was kept constant at $L = 200$ mm. The Q_u and stiffness of the single individual pile of B3L2 were about 1.47 and 1.3 times, respectively, of the corresponding values for B2L2.

Figure 5.13 Applied central point load (P) versus central settlement (δ_c) of 4- and 9-pile piled rafts with piles of the same L of 200 mm and $B = 20$ mm & $B = 30$ mm

In the 4-pile piled rafts, the larger pile width resulted in quite significant reduction in settlement when the applied load was small. The percentage reduction in δ_c increased from 20% to 40% at $P = 5$ kN and from 12% to 24% at $P = 10$ kN as the pile width was increased from 20 mm to 30 mm. However, in both cases, the benefits of the piles were obliterated as the raft approached its ultimate bearing pressure.

As noted earlier, the P - δ_c curve for raft 9PB2L2 displayed a trend which was distinctively different from the other rafts; its initial response was much more 'flexible' than even the 4-pile piled rafts when P was small. The response then became progressively 'stiffer' as P increased. Figure 5.13(a) shows that the P - δ_c curve for 9PB2L2 was similar in trend to 9PB3L2 as P exceeded 10 kN. It was therefore reasonable to compare the response of 9PB2L2 and 9PB3L2 at higher P . Figure 5.13(b) shows that the percentage settlement reduction increased from 37% to 65% when $P = 10$ kN; and from 40% to 55% when $P = 15$ kN.

In summary, increasing the pile width in the 4-pile piled rafts caused the settlement reduction to improve by about 2 fold. For the 9-pile piled raft, the improvement was nearly 1.8 times at $P = 10$ kN; and 1.4 times when $P = 15$ kN.

The P - δ_c response of the 4PB2L4, 4PB3L4, 9PB2L4, 9PB3L4 and unpiled raft are presented in Figure 5.14. This figure groups the raft equipped with piles which were 400 mm long. These piles therefore have twice the length of those shown in Figure 5.13. The load tests on single piles showed that the bearing capacity of B3L4 was about 1.6 times that of B2L4. The stiffness of B3L4 was 1.5 times stiffness of B2L4.

Figure 5.14 showed the convergence of the P - δ_c curves for both the 4-pile piled rafts and the 9-pile piled rafts when the applied load was high. Both 9-pile piled rafts showed much higher ultimate load bearing capacities when compared to the 4-pile piled rafts. At more moderate P level where the settlement reduction effect of added piles was of most interest, the benefits of the piles were clearly evident. In all cases, larger pile width has resulted in greater reduction in raft settlement, except at the initial stages when the absolute raft settlement was small. At $P = 5$ kN, increasing the pile width from 20 mm to 30 mm increased the settlement reduction from 50% to 71% for the 4-pile piled rafts and from 63% to 71% for the 9-pile piled rafts. At $P = 10$ kN, the corresponding improvement was from 32% to 57% for 4-pile piled rafts and from 56% to 69% in the 9-pile piled rafts.

Figure 5.14 Load (P) versus central settlement (δ_c) of 4- and 9-pile piled rafts with piles of the same L of 400 mm and $B = 20$ mm & $B = 30$ mm

5.4.4 Effect of pile with similar pile capacity and stiffness but different pile dimensions

The load tests on single piles B3L2 and B2L4 indicated that only small differences existed between the ultimate capacities and stiffness of these piles. The effect of pile with the same capacity and stiffness but different pile dimension on piled raft settlement was investigated by comparing the response in 4-pile and 9-pile piled rafts.

Figure 5.15 Applied central point load (P) versus central settlement (δ_c) of 4- and 9-pile piled rafts with piles of the same capacity but different

The data presented in Figure 5.15 show that the load settlement curve for the two 9-pile piled rafts were practically indistinguishable for P up to about 18 kN. When P exceeded 18 kN, the greater reduction in settlement associated with the longer and more slender piles began to become evident. However, this distinction only kicked in at loads which were higher than 0.6 times P_u of unpiled raft and hence was not directly relevant to the discussion on the use of piles to reduce settlement under working load condition.

5.4 Summary

The load-settlement behaviour of nine model piled rafts were analysed in this chapter.

The model test data showed that for a given pile size, increasing the number of piles in the raft was the primary factor that controlled the reduction of raft settlements provided the piles were spaced at a reasonable distance apart.

The test data showed that piled raft settlements were strongly influenced by the stiffness of the individual piles. Rafts supported on equal number of piles of the same capacity and stiffness showed essentially similar load-settlement response.

Long, and hence stiffer, piles reduced raft settlements more effectively and efficiently than short piles of the same width. When piles were widely placed, a two fold increase in settlement reduction was observed by increasing the length of the piles within the range of working loads.

The model test data indicated that when piles are used as settlement reducers in piled raft, they should be spaced far apart. If increasing the pile number caused the pile spacing to encroach on the optimum threshold, the expected benefit of settlement reduction might not be fully realised.

Chapter 6

Load Distribution between Raft and Piles in Piled Rafts

6.1 Introduction

For equilibrium, the applied load on a piled raft is counter-balanced by the geotechnical resistance provided by the piles and the contact pressure between the raft and soil. The piles would transfer the load through shaft friction and end bearing resistance. Due to the much stiffer load settlement characteristic of the piles as compared to raft-soil interface response, the piles would carry a larger proportion of the applied load initially as has been demonstrated in Chapter 5. As a consequence, the amount of load transferred through raft bearing pressure would be much reduced when compared to an unpiled raft subject to the same applied load. In turn, the smaller raft bearing pressure would lead to smaller settlement.

The proportion of the applied load carried by the piles and raft is not constant but changes with load level. The change in raft bearing pressure in a piled raft compared to unpiled raft varies with load level and location on the raft-soil interface. In the present study, the total pressure transducers installed at the base of the raft and the strain gauges installed in the piles allowed a direct assessment of the variation of the load sharing between raft and piles. The instrumentation enabled the monitoring of the changes in the load distribution as the applied load on the piled raft was increased.

The effect of number of piles, width of piles, length of piles and bearing capacity of piles on the load sharing between piles and raft will be examined in the following sections.

6.2 Effect of Number of Piles on the Distribution of Applied Load between Raft and Piles

In this section, the changes in the raft-soil contact pressure and the pile head loads in the piled rafts will be examined. Attention is focussed on how the number of piles in the raft, the pile lengths and the width of piles affect the mobilisation of the contact pressure and pile resistance. The term mobilisation is used to describe the changes in the resistance as the applied load (P) is increased.

Figure 6.1 Layout of piles and instrumented piles in 4-, 9- and 16-pile piled rafts

Figure 6.1 shows the layout of the piles in the rafts. Due to symmetry, all 4 piles in the 4-pile piled raft were expected to show similar behaviour and hence only 1 pile was instrumented. In the 9-pile and 16-pile piled rafts, 3 distinct piles could be identified and these were instrumented and were labelled as shown in Figure 6.1.

Mobilisation of Geotechnical Resistance of Individual Piles

Figure 6.2 shows the variation of pile head loads versus the applied load on the raft for three pile-rafts containing 4, 9 and 16 piles respectively. The piles in all the rafts were identical, 30 mm wide and 200 mm long. Figure 6.2 shows that as P increased from 0 to 9 kN, the pile in the 4-pile piled raft developed a higher pile head load when compared to piles in the 9- and 16-pile piled rafts. In turn, the piles in the 9-pile piled raft were found to attract a higher pile head load than the 16-pile piled raft

at the same P . This would suggest that the rate of mobilisation of the pile resistance would be delayed with increasing pile number in the raft. Piled raft with smaller number of piles was observed to mobilise a higher pile resistance than piled raft with a larger number of piles. This held true at least at applied load level before the pile capacities were fully mobilised.

Figure 6.2 Variation of mobilised geotechnical resistance of individual piles of $B = 30$ mm and $L = 200$ mm in 4-, 9- and 16-pile piled rafts

When the applied load was increased beyond 10 kN, the pile head loads in the 9-pile piled rafts [9PB3L2] exceeded those obtained in 4PB3L2 and 16PB3L2. The pile head loads in both 4PB3L2 and 16PB3L2 converged very closely to the capacity of B3L2 obtained from single pile load test. Hence, the difference between the 9PB3L2 and the other two rafts could be caused by other factors. One possibility is the higher c_u of the model ground used in the 9PB3L2 test. The c_u in the 9PB3L2 test was about 5% higher than the c_u in tests involving 4PB3L2 and 16PB3L2.

Within each piled raft, the pile head loads in the instrumented piles in both 9PB3L2 and 16PB3L2 also developed at different rates. At low P before the onset of

significant non-linear response, the central core labelled 1 attracted a slightly higher load than the edge piles, with the pile at the corner attracting the least load.

The near zero gradient of the curves signifies the full mobilisation of geotechnical capacities of the piles. Figure 6.2 shows that for 4PB3L2, full mobilisation of pile capacity occurred at $P = 6$ kN. For the 16-pile piled raft, it occurred at $P = 15$ kN. As discussed earlier, the slightly stronger model ground used in 9PB3L2 caused the curve to continue to trend upward.

From the load test on single piles, pile B3L2 was found to mobilise its ultimate resistance of 0.62 kN at about 0.5 mm. When the same pile was incorporated in a piled raft, it was found that Q_u was mobilised at a larger settlement. From the experiment, the full mobilisation of pile capacity occurred at 0.75 mm, 1.2 mm, and 1.5 mm for 4-pile, 9-pile and 16-pile piled rafts, respectively. The measured ultimate pile head loads in the 3 piled rafts varied between 0.55 kN to 0.70 kN. The ultimate pile capacity in the 4-pile piled raft and the centre pile in 9-pile piled raft was nearly equal to the ultimate capacity (Q_u) of the single pile B3L2 tested standalone, which was 0.62 kN. The ultimate pile head load of piles 2 and 3 in the 9-pile piled raft was about 5% and 10% higher than Q_u , respectively. In contrast, the ultimate pile head load piles 2 & 3 in the 16-pile piled raft was about 7 to 10% smaller than Q_u .

Mobilisation of Pile Resistance

The sum of the loads transmitted through piles for each of the three piled rafts is plotted against P in Figure 6.3. For P from 0 to 3 kN, the three curves overlap each other with negligible scatters. As P increased further, the 3 curves diverged, with piles in 16PB3L2 carrying a higher load. The curve for 4PB3L2 flattens out to a value of about 2.5 kN at $P = 6$ kN. The terminal value for 16PB3L2 was about 9.8 kN, a large proportion of this ultimate load was attained when P was about 12 kN. The ratio of the ultimate loads for 16PB3L2 and 4PB3L2 was slightly less than 4 - the ratio of number of piles in the two rafts. The ratio of the sum of ultimate pile loads in the 9PB3L2 and 4PB3L2 was about 10% higher than 2.25 (or 9/4, the ratio of number of piles in the two piled rafts).

Figure 6.3 Variation of mobilised total resistance by all piles with applied load on raft in 4-, 9- and 16-pile piled rafts with piles of $B = 30$ mm and $L = 200$ mm

Figure 6.4 Mobilisation of interface contact pressure with applied load at quarter points in 4-, 9- and 16-pile piled rafts with $B = 30$ mm and $L = 200$ mm

Mobilisation of Interface Contact Pressure

Figure 6.4 shows the P (or p_a)- σ_c response at the quarter points. For comparison the P - σ_c curve of unpiled raft is also shown in the same figure. The results show that the contact pressure σ_c at quarter points in all piled rafts was significantly smaller than that obtained in the unpiled raft at the same applied load P . The larger the pile number, the larger was the reduction in σ_c .

As P increased from zero to 10 kN ($p_a = 110$ kPa) in each of the piled raft, the P - σ_c curve was nearly linear indicating that the interface contact pressure was increasing at a constant rate. The gradient of the curve, or $d(\sigma_c)/d(p_a)$, was approximately 0.6, 0.5 and 0.4 for the 4-, 9- and 16-pile piled rafts, respectively. In the 9- and 16-pile piled rafts, the gradient appeared to become steeper at higher P . For $P > 18$ kN ($p_a = 200$ kPa), the gradient was about 0.9 in both 9PB3L2 and 16PB3L2. In the transition zone where $10 \text{ kN} < P < 18 \text{ kN}$, the gradient gradually increased from 0.5 and 0.4 in 9PB3L2 and 16PB3L2, respectively, to 0.9 in both rafts.

As discussed in the previous paragraphs, the piles in 9PB3L2 and 16PB3L2 mobilised their ultimate resistance when the applied load was in the range of between 10 kN to 14 kN. The piles would not contribute additional resistance when $P > 14$ kN. Thereafter, any increase in P would be resisted by increase in raft contact pressure. The constant $d(\sigma_c)/d(p_a)$ ratio of 0.9 at quarter points hinted the onset of such an event. The curve for 4PB3L2 appeared to track the unpiled raft response. This suggested the function of the piles in the 4-pile piled raft with short (200 mm length) pile was qualitatively different from the piles in the other 2 piled rafts, as alluded to earlier.

At $P = 10$ kN (or $p_a = 110$ kPa), the reduction in σ_c at quarter points was 15 kPa, 25 kPa and 35 kPa in the 4-, 9- and 16-pile piled rafts, respectively, as compared to the unpiled raft. As P was increased further, the percentage reduction in σ_c became smaller in the 4- and 9-pile piled rafts. At $P = 15$ kN ($p_a = 165$ kPa), the reduction in σ_c at quarter points was 5 kPa, 15 kPa and 40 kPa in the 4-, 9- and 16-pile piled rafts, respectively, with the unpiled raft serving as the base of reference.

Figure 6.5 shows the variation of σ_c with P (or p_a) at corner points in the 4-, 9-, 16-pile piled rafts and unpiled raft. At any applied load P , the contact pressure σ_c developed at the corners of the piled raft was smaller than that obtained in the unpiled raft. The amount of reduction in σ_c increased as the pile number in the piled-raft increased.

The general trend of variation of σ_c at the corners of the raft was broadly similar to those observed at the quarter points. At low applied load level of up to 10 kN ($p_a=110$ kPa), the gradients of the p_a - σ_c curves for the 9- and 16-pile piled rafts were approximately the same, with a value of 0.6. The gradient for the 4-pile piled raft was about 0.8. At higher load level, say for $P > 18$ kN ($p_a > 200$ kPa), the gradient of the p_a - σ_c curves in the 3 piled rafts was nearly 1. As discussed earlier, the onset of a constant gradient in the p_a - σ_c curves coincided with the full mobilisation of the pile capacities. With the proviso that a) not all the raft area was effective in transmitting the load; and b) the actual contact pressure distribution was not known with certainty, a gradient of unity for the p_a - σ_c curve meant the increase in average applied pressure was balanced by the same increase in contact σ_c .

Figure 6.5 Mobilisation of interface contact pressure with applied load at corners in 4-, 9- and 16-pile piled rafts with $B = 30$ mm and $L = 200$ mm

Using the measured contact pressure in the unpiled raft as reference, at $P = 10$ kN ($p_a = 110$ kPa), the σ_c at the corner was reduced by about 15 kPa, 20 kPa and 35 kPa for the 4-, 9- and 16-pile piled rafts, respectively. At $P = 15$ kN ($p_a = 165$ kPa), σ_c at corner points was reduced by 35 kPa and 50 kPa in the 9- and 16-pile piled rafts. For the 4-pile piled raft, a reversal in σ_c reduction to 10 kPa was observed instead.

By comparing σ_c at quarter points and corners presented in Figures 6.4 and 6.5, it was obvious that σ_c was not uniform across the raft-soil contact surface. At any given load, σ_c at corners was higher than that at quarter points. The rate of increase in σ_c was also higher at corners compared to that at quarter points. From the perspective of contact pressure reduction and with the unpiled raft serving as the reference datum, it was found that σ_c at quarter points and corner points was reduced by nearly the same amount in the 4- and 16-pile piled rafts at $P = 10$ kN. However, at a higher applied load of $P = 15$ kN, there was a greater reduction in σ_c at the corners than at the quarter points for all three piled rafts.

The above observations showed that once the piles had mobilised their full geotechnical resistance, any subsequent increase in applied load P must be counteracted by a equal increase in σ_c . The experimental results suggested that the increase in σ_c was not uniform across the raft. At quarter points, $d(\sigma_c)/d(p_a)$ was about 0.9. At the corners, $d(\sigma_c)/d(p_a)$ ratio was nearly 1. With such differential rates of change, the differences in interface contact pressure at the two locations would increase with increasing magnitude of applied load.

Variation of Raft Resistance with Applied Load

Figure 6.6 shows the variation of load transmitted via raft-soil interface pressure with P for the 4-, 9- and 16-pile piled rafts and unpiled raft. As discussed in Chapter 5, the average raft bearing pressure was estimated by assuming that σ_c measured at quarter points and at the corners were effective over $2/3$ and $1/3$ of the raft area, respectively. The total load transmitted via raft-soil interface pressure was computed by multiplying the estimated average raft bearing pressure with the net raft-soil contact area. The net area was computed by subtracting the area occupied by the piles from the gross raft area.

Figure 6.6 Variation of raft bearing resistance with applied load in 4-, 9- and 16-pile piled rafts with $B = 30$ mm and $L = 200$ mm

Figure 6.7 Variation of percentage load carried by piles and raft with the applied load (P) in 4-, 9- and 16-pile piled raft with piles of $B = 30$ mm and $L = 200$ mm

The load transmitted through raft-soil interface contact pressure was termed raft bearing resistance in this report. Figure 6.6 shows that at the same applied load P the raft bearing resistance in the piled rafts was much smaller than that in the unpiled raft. At $P = 10$ kN, the ratio of raft bearing resistance to applied load was approximately 0.7, 0.5 and 0.4 respectively for the 4-, 9- and 16-pile piled rafts. In absolute terms, the raft bearing resistance of the 4-pile piled raft was reduced by about 1.8 kN when compared to unpiled raft. In the 9- and 16-pile piled rafts, the reduction was 3.5 kN and 4.5 kN, respectively.

The curves of raft bearing pressure exhibited a final linear segments with constant gradients at high applied load level, say for $P > 18$ kN. The linear segments were nearly parallel, sharing a common gradient of approximately 0.85. In an ideal scenario where the only source of counteracting forces were pile resistance and raft-soil contact pressure, the gradient of the slope should be unity as the piles would have fully mobilised their capacities at this stage and could no longer provide additional support. In reality, however, there were significant frictional resistance between the sides of the raft and the surrounding soil, particularly when large raft settlement had occurred. The assumed average bearing pressure also might not reflect accurately the actual distribution of the contact pressure. A gradient of 0.85 could imply that errors due to these various factors might be as high as 15% of the applied load.

Load Distribution between Raft and Piles

Figure 6.7 depicts an evolving picture of how the applied load was distributed to the piles and raft as P was increased. The vertical axis shows the load carried by the piles and raft expressed as a percentage of the applied load. The curve representing the load carried by piles shows a distinct peaks. These peaks occurred at different P , with a clear trend that as the pile number increased, the maximum percentage load carried by piles would occurred at higher P . At the peak, the piles in the 4-pile piled raft carried 58% of the applied load; when $P = 2$ kN. In the 9-pile piled raft, the maximum contribution from piles was 58% which occurred at $P = 5.5$ kN. In the 16-pile piled raft, the peak contribution from piles was 82% when $P = 9$ kN.

Post peak, the percentage load carried by piles decreased gradually as P was increased; this was compensated by an increase in the raft bearing resistance. In 4PB3L2, the percentage of applied load carried by piles was reduced to 16% when $P = 23$ kN. For 9PB3L2 and 16PB3L2, the corresponding values were 30% and 41%, respectively.

The curves representing the pile load and raft bearing resistance for 16PB3L2 (denoted by solid and open square symbols) are particularly illuminating. As the pile load increased from $\sim 58\%$ to a maximum of $\sim 82\%$ at $P = 9$ kN, the raft load was found to mirror the changes, decreasing from a high of about 44% initially to a minimum of $\sim 24\%$ at $P = 9$ kN. For $P > 9$ kN, the reduction in pile load contribution was accompanied by a corresponding increase in the raft bearing resistance.

The sum of the loads carried by the piles and raft did not exactly equal the applied load; i.e. they did not sum up to 100% exactly. The amount of discrepancy fluctuated with P . The sum initially exceeded P when P was small, but underestimated P when P was large. The load at which the transition from overestimation to underestimation occurred was not the same for the three piled-rafts. It happened at $P = 7.5$ kN, 12kN and 15kN for the 4-, 9-, and 16-pile piled rafts, respectively. The maximum overshoot was observed in the 16-pile piled raft, where the sum exceeded the applied load by about 10% when $P = 10$ kN. The maximum shortfall was about 20% which occurred in the 4-pile piled raft when $P > 18$ kN. As postulated in Chapter 5, the source of the discrepancy could be attributed to frictional resistance mobilised at the vertical sides of the raft and errors in the assumed pressure distribution.

Piles Raft with Long Piles ($L = 400$ mm)

The results presented in the preceding sections are for piles which were 200 mm in length. It was noted that at high P , the response of the 4-pile piled rafts equipped with short piles ($L = 200$ mm) were dominated by the bearing capacity failure mechanism associated with the raft. The short piles were not effective as they were

engulfed entirely within the failure soil zone. The behaviour of piled raft fitted with 400 mm long piles would be examined in the following sections.

Mobilisation of Geotechnical Resistance of Individual Piles

Figure 6.8 shows the variation of pile head loads with P in 4PB3L4 and 9PB3L4. The piles in these rafts were 30 mm wide and 400 mm long. The magnitude of the pile head load at any instant was a reflection of the mobilisation of the geotechnical resistance in the individual pile. In contrast to the results for the shorter piles shown in Figure 6.2, the curves for the long piles are more tightly banded and show smaller scatters. The pile head load in the central piles of 9PB3L4 exhibited broadly similar variation with P as the piles in 4PB3L4. At higher P , the central pile (pile 1) in 9PB3L4 attracted a slightly higher pile head load larger than the pile in 4PB3L4. At any given P , pile 1 in 9PB3L4 was found to show a higher pile head load than pile 2 (mid-edge) and pile 3 (corner).

Figure 6.8 Applied central point load (P) versus mobilised geotechnical resistance of piles of $B = 30$ mm and $L = 400$ mm in 4- and 9-pile piled rafts

The pile in 4PB3L4 and pile 1 in 9PB3L4 mobilised their maximum resistance at $P = 15$ kN. In pile 2 and pile 3 of 9PB3L4, the pile resistance continued to increase at a very gradual rate even when $P > 20$ kN. The ultimate resistance mobilised in 4PB3L4 and pile 1 in 9PB3L4 was found to be about 5% to 10% smaller than Q_u obtained from load test on a single B3L4 pile.

Figure 6.9 shows the sum of the pile head loads in 4PB3L3 and 9PB3L4. The data shows that the ultimate resistance of piles in 4PB3L4 was mobilised at an earlier stage than the piles in 9PB3L4. More than 90% of the ultimate pile resistance was activated in 4PB3L4 when $P = 10$ kN. In contrast, the piles in 9PB3L4 only mobilised 90% of its ultimate resistance at $P = 18$ kN. As to be expected, the sum of the ultimate pile resistance was related to the number of piles in each raft. The ratio of the ultimate pile resistance in the 9-pile piled raft and the 4-pile piled raft was about 2.1, slightly smaller than 2.25, the ratio of number of piles in the two rafts.

Figure 6.9 Applied central point load (P) versus mobilised geotechnical resistance by all piles in 4- and 9-pile piled rafts with piles of $B = 30$ mm and $L = 400$ mm

Mobilisation of Interface Contact Pressure

Figure 6.10 shows the σ_c versus P (or p_a) response at quarter points of 4PB3L4, 9PB3L4 and unpiled raft. The response was broadly similar to those of rafts fitted with 200 mm piles shown in Figure 6.4. The increase in pile length had led to a greater reduction in σ_c . Similar to the results for the short piles, the curves show a largely linear response with P initially, this is followed by a transition zone, and finally a linear segment at higher P . For P up to 10 kN, the gradient of the curve, or $d(\sigma_c)/d(p_a)$ ratio, was about 0.6 and 0.4 for 4PB3L4 and 9PB3L4, respectively. The final linear section of the curves began at $P > 15$ kN for 4PB3L4 and at $P > 19$ kN for 9PB3L4. At these load levels, the piles would have mobilised more than 90% of their capacities. The gradient of the curves ($d(\sigma_c)/d(p_a)$) was 0.8 and 0.75 for 4PB3L4 and 9PB3L4, respectively. This meant that 80% and 75% of the applied average pressure was transmitted through interface contact pressure in the two rafts; the same caveat of uncertainties concerning the net contact area and the assumed pressure distribution stated previously still applied.

Figure 6.10 Mobilisation of interface contact pressure (σ_c) with applied load (P) at quarter points in 4- and 9- piles rafts with $B = 30$ mm and $L = 400$ mm

Figure 6.11 shows the σ_c versus p_a response at the raft corners for 4PB3L4, 9PB3L4 and unpiled raft. For P up to 10 kN, $d(\sigma_c)/d(p_a)$ at the corners was about 0.6 and 0.4 in 4PB3L4 and 9PB3L4 respectively. These values were similar to those obtained at the quarter points. This meant the interface contact pressure was being mobilised at the same rate at these two locations. However, for $P > 18$ kN, $d(\sigma_c)/d(p_a)$ was approximately 1 in 4PB3L4 and about 0.9 in 9PB3L4. A value of 1 for $d(\sigma_c)/d(p_a)$ meant the interface contact pressure was increasing at the same rate as the increase in average applied pressure. The higher $d(\sigma_c)/d(p_a)$ values at the corner points would cause the σ_c there to increase at a faster rate at the corners than at the quarter points.

Figure 6.11 Mobilisation of interface contact pressure (σ_c) with applied load (P) at corners in 4- and 9- piles rafts with $B = 30$ mm and $L = 400$ mm

Load Distribution between Piles and Raft

Figure 6.12 shows the variation of raft bearing resistance with P . Initially, as P increased from zero, the raft bearing resistance in the 9PB3L4 increased rather slowly when compared to the 4PB3L4. This was because initially the piles in

9PB3L4 were carrying a higher percentage of the applied load. The gradient of the curves, which measures the rate of increase of raft bearing resistance with applied load, increased with P in both cases. For $P > 15$ kN, the curve for 4PB3L4 become linear with a gradient of 0.9. For 9PB3L4, the linear segment began at $P = 20$ kN and has a gradient of ~ 0.75 .

Figure 6.13 depicts the evolution of raft bearing resistance and total pile loads expressed as a percentage of applied load P . The graph for 4PB3L4 decreased monotonically as P was increased, the maximum pile contribution of 55% occurred at $P = 2$ kN. It then diminished to a value of less than 20% when $P = 25$ kN. For 9PB3L4, the percentage of applied load carried by piles increased from 58% initially to a maximum of 80% at $P = 5$ kN; it then reduced to a low of 40% at $P = 25$ kN. The trend shows that it would continue to decrease but at a much diminished rate beyond $P = 25$ kN.

Figure 6.12 Variation of raft bearing resistance with applied load in 4- and 9-pile piled rafts with $B = 30$ mm and $L = 400$ mm

Similar to the piled raft with the shorter B3L2 piles, the curve for load carried by piles in 9PB3L4 showed a distinct maximum of 82% when $P \approx 5$ kN. The pile contribution then diminished gradually to 36% when $P \approx 28$ kN. No peak was discernable for 4PB3L4, with the pile contribution reduced monotonically from

55% at $P = 2$ kN to less than 16% at large P . The load carried by the raft in 9PB3L4 varied from a minimum of $\sim 28\%$ at small P to about 50% at $P = 28$ kN. For 4PB3L4, the raft contributed 46% to 77% of the applied load as P was increased.

The maximum imbalance between the applied load and mobilised resistance in 4PB3L4 was about +12% to -8%. In 9PB3L4, the maximum imbalance between the applied load and mobilised resistance was about +12% to -15%.

Figure 6.13 Percentage load sharing between piles and raft in 4- and 9-pile piled rafts with $B = 30$ mm and $L = 400$ mm

6.3 Effect of Piles Length on Distribution of Applied Load between Raft and Piles

Pile length would affect both the ultimate resistance and the initial stiffness (gradient of the load-displacement curve) of the piles. This section will examine the effect of varying pile length on the load-transfer mechanism on the 4-pile and 9-pile

piled rafts. The mobilisation of pile 2 in the 9-pile piled raft was found to be intermediate between piles 1 and 3 and would be adopted for analysis. The results for piles 1 and 3 are presented in Appendix B.

The piles used in the rafts were 20 mm wide. Piles of length 200 mm (B2L2) and 400 mm (B2L4) were used in each of the two 4-pile and 9-pile piled rafts, respectively. From the load tests on single piles, the ultimate bearing capacity of pile B2L4 was found to be 1.6 times B2L2. The initial stiffness response of B2L4 was 1.3 times pile B2L2.

Mobilisation of Geotechnical Resistance of Individual Piles

The results from the four model piled rafts tests are plotted in Figure 6.14. The test data showed that the longer piles developed higher pile resistance in both the 4-pile and 9-pile piled rafts. The difference was particularly pronounced in the 4-pile piled rafts (represented by triangular symbols) labelled 4PB2L2 and 4PB2L4. Due to the lower pile capacity associated with shorter pile length, the full capacities of the piles in 4PB2L2 were mobilised slightly earlier than the longer piles in 4PB2L4: at $P = 8.5$ kN in the former and at $P = 12$ kN in the latter.

It was observed that for the same pile length, the piles in the 4-pile piled raft tended to mobilise its capacity earlier than the piles in the 9-pile piled raft. This was to be expected as the applied load was shared amongst a larger number of piles; each pile would carry a smaller load. This observation would suggest that, from the perspective of a better utilisation of pile to control raft settlement, a small number of longer, stiffer high capacity piles is more effective than the deployment of a large number of softer, small capacity piles.

For the same pile type in both 4-pile and 9-pile piled rafts, the ultimate loads attained were found to be not significantly different from the capacities obtained for single piles. The ratio of the ultimate pile resistance mobilised in the raft equipped with B2L4 and B2L2 was about 1.7.

Figure 6.14 Applied central point load (P) versus mobilised geotechnical resistance of piles of $B = 20$ mm and $L = 200$ mm & 400 mm in 4- and 9-pile piled rafts

Figure 6.15 Applied central point load (P) versus mobilised geotechnical resistance by all piles in 4- and 9-pile piled rafts with piles of $B = 20$ mm and $L = 200$ & 400 mm

Mobilisation of Pile Resistance

Figure 6.15 shows the variation of the sum of the pile loads with P . The ultimate total pile resistance was found to be equal to the product of the number of piles and the ultimate capacity of single pile.

Mobilisation of Interface Contact Pressure

The development of interface contact pressure in the four piled rafts was generally similar to those presented in Section 6.2. In this section, only the raft bearing resistance would be discussed. The individual contact pressure responses are included in Appendix B. The raft bearing resistance was computed in the same manner described in Section 5.2. The variations of mobilised bearing resistance for 4- and 9-pile piled rafts with type B2L2 and B2L4 piles are shown in Figure 6.16.

The mobilised raft resistance in all four piled rafts was significantly smaller than the unpiled raft; particularly at the initial stages. The ratio of mobilised bearing resistance to applied load was about 0.7 and 0.6 for 4PB2L2 and 4PB2L4, respectively. In contrast, in the unpiled raft, the ratio was slightly greater than 1 for $0 < P < 5$ kN. In both 4-pile and 9-pile piled rafts, the 400 mm piles had led to a greater reduction in raft bearing resistance when compared to the shorter, 200 mm piles. The effect of pile length was also more pronounced in the 4-pile piled raft than in the 9-pile piled raft. This was counter-intuitive as one would expect that with more than double the number of piles, the 9-pile piled raft should exert a greater impact on the raft bearing resistance. The observed behaviour could be explained by the faster rate of mobilisation of pile resistance in the 4-pile piled raft compared to the 9-pile piled rafts. The results thus confirmed the hypothesis that for the purpose of reducing the bearing pressure and hence raft settlement, it would be more effective to deploy the required pile resistance by using a small number of long, stiff piles spaced far apart to take advantage of the faster rate of mobilisation and a better utilisation of the pile capacities, than the alternative of using a large number of smaller capacity piles.

Figure 6.16 Variation of raft bearing resistance with applied load in 4- and 9-pile piled rafts with piles of $B = 20$ mm and $L = 200$ & 400 mm

Load Distribution between Piles and Raft

The variation of raft bearing resistance and total mobilised pile resistance with applied load P is plotted in Figure 6.17. The mobilised raft and pile resistances were expressed as a percentage of the applied load. The piles in 4PB2L4 contributed more than 70% of the total applied load initially, reducing to less than 18% when P was high. For 4PB2L2, the corresponding values were 37% and less than 18%, respectively

The response in the 9-pile piled raft was more complex. The contribution from piles load in 9PB2L2 varied from 50% of the applied load initially, increasing to a maximum of 58% at $P = 8$ kN, and thereafter reduced to less than 20% at high P . In 9PB2L4, the pile load contribution made up about 50% of the applied load initially, but it then monotonically decreased to less than 20% at large P . Figure 6.17 showed that raft equipped with longer piles tend to carry a higher percentage of the applied load, particularly at moderate applied load level.

Figure 6.17 Percentage load sharing between piles and raft in 4- and 9-pile piled rafts with piles of $B = 20$ mm and $L = 200$ mm and 400 mm

6.4 Effect of Pile with Similar Pile Capacity and Stiffness but Different Geometry on Distribution of Applied Load between Raft and Piles

The effect of increasing pile length has been illustrated in Section 6.3. However, as piles of the same cross-section and different pile length would have different load bearing capacities, the effect of pile length and bearing capacity could not be clearly differentiated.

As discussed in Section 4.3, the load tests on isolated single B3L2 and B2L4 piles had indicated that only small differences existed between the ultimate bearing capacities of these piles. Apart from the differences in length, these piles differed in width. Moreover, the load test showed that the bearing stiffness of piles B3L2 and B2L4 were very similar. Hence, an examination of the behaviour of pile rafts

equipped with these two types of piles could provide better insight into the effect of pile length.

Mobilisation of Geotechnical Resistance of Individual Piles

The variations of the mobilised total pile resistance and raft bearing resistance with P are shown in Figures 6.18 and Figure 6.19 for four piled rafts. Each of the two 4-pile and 9-pile piled rafts were installed with B3L2 (width = 30 mm, length = 200 mm) and B2L4 (width = 30 mm and length = 400 mm) piles, respectively.

Figure 6.18 shows that for the 4-pile piled rafts, both the B3L2 and B2L4 piles mobilised the geotechnical resistance at the same rate for P up to 7 kN. The initial response was largely linear. Similar behaviour was observed also for piles in 9-pile piled rafts, except that the geotechnical resistance was mobilised at a slower rate when compared to the 4-pile piled rafts. This was primarily due to the availability of a larger number of piles in the 9-pile piled rafts to share the applied load. Again the initial response was linear and persisted up to $P = 10$ kN.

Figure 6.18 Load (P) versus mobilised geotechnical resistance of piles of $B = 30$ mm & $L = 200$ mm and $B = 20$ mm & $L = 400$ mm in 4- and 9-pile piled rafts

Figure 6.19 Variation of raft bearing resistance with applied load in 4- and 9-pile piled rafts with piles of $B = 30$ mm & $L = 200$ mm and $B = 20$ mm & $L = 400$ mm

The differences in the mobilisation of the pile capacity in a piled-raft would have important implication for pile selection in reducing raft settlement. The earlier mobilisation of pile capacity in the smaller pile groups would imply a more effective use of the pile capacities, particularly if the applied load was low. The full mobilisation of pile capacities at lower applied load when a smaller number of piles was used in the raft meant better utilisation of the available pile capacity.

For the 4-pile piled rafts at higher P , the pile resistance mobilised in 4PB2L4 was somewhat higher than that observed in 4PB3L2, possibly caused by the differences in the undrained shear strength of the foundation soils used in the model tests. For the 9-pile piled rafts, the two pile types were observed to yield the same ultimate resistance at high P .

Raft Bearing Resistance

The two curves representing the variation of raft bearing resistance with P in the 9-pile piled rafts equipped with B3L2 and B2L4 piles showed only minor differences.

The difference in the 4-pile piled rafts were slightly bigger, with 4PB3L2 showing a slightly larger bearing resistance which did not appear to vary significantly with applied load P . The data suggest that for a given piled raft, piles of different dimensions but with the same pile capacity and stiffness would alter the piled raft in similar manners, particularly with respect to the load distribution between piles and raft.

Further evidence was observed in the variation of load distributions between piles and raft with P as shown in Figure 6.20. Except for the deviations in the percentage contribution due to pile resistance for $P < 7$ kN, the two pile types B3L2 and B2L4 gave rise to similar response in the 9-pile piled rafts. The differences in the bearing resistance response for the 4-pile piled rafts could be attributed to the differences in the undrained shear strength of the foundation soils. The higher pile resistance and stiffness in 4PB2L4 had accentuated the greater effectiveness of the long piles in reducing raft bearing pressure and hence raft settlement.

Figure 6.20 Percentage load sharing between piles and raft in 4- and 9-pile piled rafts with piles of $B = 30$ mm & $L = 200$ mm and $B = 20$ mm & $L = 400$ mm

6.5 Summary

In a piled raft, the applied load is carried by the soil resistance provided by the piles and the raft bearing resistance. Due to the much stiffer load settlement characteristic of the piles as compared to soil bearing response at the raft soil interface, the piles would initially carry a larger proportion of the applied load and transferred it to deeper soil strata. As a result, the amount of load transferred through raft bearing pressure would be much smaller when compared to an unpiled raft subject to the same applied load. This would lead to smaller settlement in the piled raft. The distribution of load between raft and piles was not constant but changed with load level. The raft-soil contact pressure and its spatial variation would also vary with load level.

Mobilisation of Geotechnical Resistance of Individual Piles & Total Pile Resistance

The experimental results showed that the rate of mobilisation of soil resistance of individual piles with applied load is dependent on the pile number, pile length and pile width. Furthermore, it was found in a piled raft, piles formed of different length and width but having the same capacity and stiffness would mobilise their resistance at the same rate.

Increasing the pile number in a raft would reduce the rate of mobilisation of geotechnical resistance of the individual piles. Thus, for a given applied load, the resistance mobilised by individual piles in a piled raft with smaller number of piles would be higher than the resistance mobilised by individual piles in a piled raft with a larger number of piles. This was true at applied load level before the pile capacities were fully mobilised.

In the raft with the same number of piles, at low to moderate load level, long piles tended to provide higher pile resistance than short piles. Hence, the use of a small number of longer, stiffer piles is more effective in reducing pile raft settlement than the deployment of a large number of small shorter piles.

In all of the model piled rafts, the ultimate resistances mobilised by the individual piles were, for all practical purpose, equal to the ultimate resistances of the corresponding single piles.

Mobilisation of Interface Contact Pressure & Raft Bearing Resistance

The inclusion of piles in a raft significantly reduced the development of interface contact pressure with applied load. For an applied load that did not cause pile failure, increasing the number of piles in the raft led to a greater reduction in σ_c . For a given number of piles, longer and stiffer pile resulted in a greater reduction in σ_c .

In all nine piled rafts tested, the applied load versus raft bearing resistance curves showed a characteristic response comprising 3 distinct segments. The initial segment was approximately linear with a relatively gentle slope. This was followed by a transition zone with varying gradient which corresponded to the onset of nonlinearity in pile resistance. Finally, the full mobilisation of pile resistance was accompanied by the emergence of a final linear segment in the raft bearing resistance curve. This last linear segment had a steeper gradient equal to the corresponding gradient in the unpiled raft.

The gradient of the initial linear segment was strongly influenced by the mobilised pile resistance. The greater the pile resistance mobilised, the smaller would be the initial slope. In consequence, the initial slope of the raft bearing resistance curve was a function of the pile number, pile length and pile width in the piled raft.

Chapter 7

Development of Interface Contact Pressure and Piles Response in Piled Raft

7.1 Introduction

In Chapter 6, the mobilisation of interface contact pressure with applied load was discussed. From the experimental data it was found that in a piled raft, the pile number, pile length and pile width would affect the mobilisation of interface contact pressure with applied load. Chapter 5 shows that piled raft settlement was also affected by the same set of parameters. In this chapter the relationship between interface contact pressure and raft displacement would be examined. The test data presented in Chapter 6 show that the load-settlement response of a piled raft was critically dependent on the rate of mobilisation of geotechnical resistance of piles. In this chapter, the stiffness or load-displacement relationship of individual piles in the raft would be examined. These data would provide a direct measure of the soil spring and pile spring parameters. The information is important for simplified piled raft analysis in which the response of the ground and piles are represented by equivalent soil and pile springs.

7.2 Mobilisation of Interface Contact Pressure with Raft Displacement

In this section, the effect of various pile parameters on the development of interface contact pressure with raft displacement would be presented. The data were obtained from two locations on the raft-soil contact surface: the quarter point and the corner point (see Figure 3.7). The contact pressure was measured by total stress transducers and the corresponding displacements were measured by LVDTs positioned on the top surface of the raft directly above the transducers.

Mobilisation of interface contact pressure at quarter points

Figure 7.1 shows the changes in measured interface contact pressure (σ_c) with raft displacement (δ) at quarter points in the four 4-pile piled rafts and unpiled raft. Similar to the unpiled raft, the σ_c versus δ curve was non-linear even at small δ . The σ_c - δ curves for the piled rafts and the unpiled raft were tightly banded with negligible scatters. The test results suggested that in the 4-pile piled rafts, changes in length and width of the pile did not have a significant effect on the mobilisation of interface contact pressure with raft displacement.

Figure 7.1 Mobilisation of interface contact pressure (σ_c) with settlement (δ) at quarter points in first load cycle in all 4-pile piled rafts

The σ_c - δ curves at quarter points for all 9-piled rafts is compared with the unpiled raft in Figure 7.2. Initially, up to $\sigma_c = 80$ kPa, all the σ_c - δ curves for the 9-piled rafts and the unpiled raft overlapped each other. For $\sigma_c > 80$ kPa, the curves started to diverge. For $\sigma_c > 80$ kPa, the curves of the piled rafts manifested stiffer response than the unpiled raft. Although minor scatters were observed in the curves for the 9-

piled rafts, the differentiation between piled and unpiled rafts was very distinct when the contact pressure exceeded 80 kPa.

Figure 7.2 Mobilisation of interface contact pressure (σ_c) with settlement (δ) at quarter points in first load cycle in all 9-pile piled rafts

Figure 7.3 Mobilisation of interface contact pressure (σ_c) with settlement (δ) at quarter points in first load cycle in 4-, 9- and 16-pile piled rafts with piles of $B = 30$ mm and $L = 200$ mm

Figure 7.3 shows the σ_c - δ curves of 4-, 9- and 16-pile piled raft with pile B3L2 with $B = 30$ mm and $L = 200$ mm and the unpiled raft. In all these cases, the initial σ_c - δ response curves overlapped each other for σ_c up to 80 kPa. For $\sigma_c > 80$ kPa, the σ_c - δ curves for 4-pile piled raft continued to track the unpiled raft response very closely. The σ_c - δ curves for 16PB3L2 and 9PB3L2 were similar and stiffer than the response in the unpiled raft.

Mobilisation of Interface Contact Pressure at Corners

The σ_c - δ curves at the corners of all the four 4-pile piled raft and unpiled raft are presented in Figure 7.4. The data indicated that the σ_c versus δ response at corners in 4PB2L2 and 4PB3L2 was similar to that of the unpiled raft up to $\sigma_c = 180$ kPa. For $\sigma_c > 180$ kPa, larger δ values were obtained for rafts with short piles, 4PB2L2 and 4PB3L2 when compared with unpiled raft.

Figure 7.4 Mobilisation of interface contact pressure (σ_c) with settlement (δ) at corners in first load cycle in all 4-pile piled rafts

In the rafts with long piles, 4PB2L4 and 4PB3L4, the σ_c - δ curves were similar to that of unpiled raft for contact pressure up to $\sigma_c = 120$ kPa. For $\sigma_c > 120$ kPa, 4PB2L4 and 4PB3L4 showed much higher δ than unpiled raft.

Figure 7.5 shows the σ_c - δ response for the four 9-pile piled rafts and unpiled raft. As has been observed in all the piled rafts examined so far, the initial σ_c versus δ responses in all the 9-pile piled rafts were similar to the response in the unpiled raft for σ_c up to 120 kPa. For $\sigma_c > 120$ kPa, the curves began to diverge. The largest divergence was observed in the curves for rafts equipped with long piles, 9PB2L4 and 9PB3L4.

Figure 7.5 Mobilisation of interface contact pressure (σ_c) with settlement (δ) at corners in first load cycle in all 9-pile piled rafts

Taken together with the test data obtained for the 4-pile piled raft, the results suggested that the σ_c - δ relationship at the raft-soil interface was not particularly sensitive to pile parameters when σ_c and δ were small. At larger raft displacement,

the σ_c - δ curves of rafts with short piles ($L = 200\text{mm}$) did not deviate significantly from the unpiled raft. However, rafts fitted with long piles ($L = 400\text{ mm}$) gave rise to σ_c - δ curves which were distinctly different from the unpiled raft when σ_c become large. More specifically, the σ_c - δ curve tended to shift to the right of the graph for unpiled raft as pile length and pile width increased.

Differential Rate of σ_c - δ Development at Quarter Points and Corner

In Figure 7.6, the σ_c - δ response at raft corners is compared with the response at quarter points for unpiled raft, 9PB3L2 and 16PB3L2. Based on the discussion so far, one would expect the σ_c - δ curves at the corners and quarter points to be nearly identical at low contact pressure level. Figure 7.6 confirms this, showing only minor differences in the response at quarter points and corners for σ_c up to $\sim 40\text{ kPa}$. For $\sigma_c > 40\text{ kPa}$, the σ_c - δ curves at quarter points and corners began to diverge. The divergence was observed in the test data for both the piled and unpiled rafts. The rate of increase of δ with σ_c was higher at quarter points than at the corners. The quarter points response of the two piled rafts was nearly identical and stiffer than the unpiled raft response. In contrast, the response at the corners for the two piled rafts shows appreciable difference, and both were initially ‘softer’ than the unpiled raft response until δ exceeded 7 mm . However, the trend of the graph at the corner of the unpiled raft was qualitatively different from the other graphs, with steep gradient even at high raft displacement. The cause of this observed anomaly was not known.

Observation

The results from the model tests suggest that the contact pressure-displacement or σ_c - δ relationship at the raft-soil interface was non-linear, even when δ and σ_c were small. The σ_c - δ relationship at the corner of the raft was stiffer than that at the quarter points. However, a unique relationship appeared to exist for the response at the corner and quarter points. For low to moderate σ_c level of about 60 kPa , the existence of piles in the raft and the pile dimensions had only small effects on the σ_c - δ curve. Hence, in practical application the use of a unique soil spring curve

might suffice, particularly when the stress level remained was not high. At high stress levels, the 'soil spring' response at the corner and quarter points began to diverge and hence separate 'soil spring' curves should be adopted for different locations on the raft. The tests data suggested that the introduction of piles tended to cause a 'softening' of the 'soil spring' curve at quarter points of the unpiled raft; and a 'stiffening' of the soil spring curve at the corner of the unpiled raft.

In the simplified numerical analysis, in addition to adopting the 'soil spring' determined from the experiments, the interactions between the 'soil springs' need to be accounted for in an appropriate manner. The soil spring stiffness determined in the experimental study inevitably includes the bending rigidity of the raft. For adopting the soil spring stiffness in the simplified analysis, if the rigidity of the raft to be modelled is different from the raft used in the experiment, the effect of raft rigidity has to be accounted for in suitable way.

Figure 7.6 Mobilisation of interface contact pressure (σ_c) with settlement (δ) at quarter points and corners in first load cycle in the unpiled raft, 9PB3L2 and 16PB3L2

7.3 Load-Settlement Behaviour of Piles in the Piled Raft

In this section the load-settlement behaviour of piles in the piled raft would be examined with respect to the response of the corresponding single piles. Figure 7.7 shows the pile head load versus displacement responses of B3L2 pile tested in isolation, and when it was incorporated in the 4- and 9-pile piled rafts. The data showed that the initial gradient of the load-settlement response of piles in 4-pile piled raft and 9-pile piled raft were considerably smaller than that of the isolated pile. The pile in 4PB3L2 and the corner pile (pile 3) in 9PB3L2 showed similar response until the settlement reached 0.4 mm. For $0.4 \text{ mm} < \delta < 0.8 \text{ mm}$, the pile head load mobilised by pile in 4PB3L2 was higher than the corner pile in 9PB3L2. The pile in 4PB3L2 had fully mobilised its ultimate resistance at $\delta = 0.6 \text{ mm}$ and remained unchanged thereafter. In contrast, the load in the corner pile of 9PB3L2 continued to increase, albeit with a much reduced gradient beyond a displacement of 0.6 mm; it attained the ultimate load only at $\delta = \sim 3 \text{ mm}$. At any given settlement, the pile head loads mobilised by pile 2 (mid-edge) and pile 3 (centre pile) were smaller than the load in the corner pile.

Figure 7.7 Pile head load versus pile head displacement response of pile B3L2 ($B = 30 \text{ mm}$ & $L = 200 \text{ mm}$) in single individual case, 4- and 9-pile piled rafts

The ultimate pile head load mobilised by all piles in 9PB3L2 was higher than that of single individual pile; they were also slightly larger than the ultimate load attained by the pile in 4PB3L2. Such variation in the ultimate loads could be caused by differences in the undrained shear strengths of the foundations soils.

Figure 7.8 shows the pile head load versus displacement response of pile B3L2 when tested as a standalone pile and when incorporated in a 16-pile piled raft. The stiffness of the piles in 16PB3L2 was smaller than that of the single pile. Before the onset of significant nonlinearity, the corner pile was found to attract the highest load and the interior pile 1 attracting the least load. As the ultimate capacities become fully mobilised, the difference become negligible; the magnitude of the mobilised ultimate loads of the piles in the raft was slightly smaller than that of the single pile.

Figure 7.8 Pile head load versus pile head displacement response of pile B2L3 ($B = 30$ mm & $L = 200$ mm) in single individual case and 16-pile piled rafts

Figure 7.9 shows the pile head response of pile B2L2 when tested as a single pile and when it was incorporated in the 4PB2L2 and 9PB2L2 rafts. Significant reduction of pile stiffness in the raft was observed; the gradient of the curves for the piles in 4PB2L2 and 9PB2L2 was considerably smaller than the single pile. In

9PB2L2, the central pile shows a much softer response than the corner and edge piles. At large displacement, all piles yielded ultimate pile head loads which were not significantly different from the ultimate capacity of the single pile.

Figure 7.9 Pile head load versus pile head displacement response of pile B2L2 ($B = 20$ mm & $L = 200$ mm) in single individual case, 4- and 9-pile piled rafts

The data presented in Figures 7.7 to 7.9 show that pile stiffness was significantly affected by the number of piles present in the raft. Specifically, increasing the pile number in the raft tended to cause a greater reduction in pile stiffness. It should be pointed out that the dimension of the raft was fixed in this study, and hence increasing pile number reduced the spacing between piles. Therefore, the changes in the pile stiffness could also reflect the complex soil-pile interaction at different pile spacings.

Effects of pile lengths

Figures 7.10 and 7.11 show two additional sets of test results for 4-pile and 9-pile piled rafts in which longer piles were used. Figure 7.10 shows the results for rafts fitted with piles which were 20 mm wide and 400 mm long (B2L4) and Figure 7.11 shows the results for raft fitted with piles which were 30 mm wide and 400 mm long (B3L4). The single pile results were also shown in the figures for comparison.

By comparing the results shown in Figure 7.9 and Figure 7.10, it is clear the pile B2L4 was much stiffer than pile B2L2. At the same pile spacing, the longer piles experienced a smaller reduction in stiffness when it was deployed in the piled rafts. For δ less than 0.2 mm, the responses of various piles could not be clearly differentiated. At higher pile head displacement, it became apparent that piles in the 9-pile piled raft exhibited a more flexible response than those in the 4-pile piled raft.

Figure 7.10 Pile head load versus pile head displacement response of pile B2L4 ($B = 20$ mm & $L = 400$ mm) in single individual case, 4- and 9-pile piled rafts

Figure 7.11 shows that for pile B3L4 pile (30 mm wide and 400 mm long), the response of the piles in both the 4-pile piled and 9-pile piled raft for $\delta < 0.4\text{mm}$ was practically identical. The stiffness was slightly smaller than the single pile response. There were some scatters in the ultimate capacities of the individual piles, with a mean value about 10% smaller than the single pile capacity.

Figure 7.11 Pile head load versus pile head displacement response of pile B2L4 ($B = 30\text{ mm}$ & $L = 400\text{ mm}$) in single individual case, 4- and 9-pile piled rafts

In summary, the test data show that pile B2L2 with the shortest length and smallest width experienced the greatest reduction in pile stiffness when it was incorporated in the raft. The largest pile stiffness reduction occurred in the centre pile of 9PB2L2. In the case of the 400 mm long piles, the pile stiffness was moderately affected when they were employed in the rafts. At small displacement, only negligible differences in pile stiffness in both the 4-pile and 9-pile piled rafts were observed.

7.4 Evaluating Effectiveness of Piles in Reducing Raft Settlement

In an unpiled raft, the applied load is resisted by the bearing pressure developed at the raft-soil interface. The mobilisation of soil pressure at the raft-soil interface is governed primarily by soil stiffness. Typically, significant displacement must occur to mobilise sufficient bearing resistance, leading to large raft settlement. The unpiled raft test result indicated that at a load of 10 kN, the raft displacement was about 3 mm.

The single pile load tests showed that pile capacities, particularly shaft resistance, were mobilised at very small relative pile-soil displacement. For the four model piles, the capacities (shaft + end bearing) were fully mobilised at a pile head displacement of less than 1 mm as shown in Figures 4.21 & 4.22.

In the case of a piled raft, the applied load was supported by both the raft bearing pressure and pile resistance. The stiffness and capacities of the piles and the nature of the raft-soil interface strongly influenced the distribution of the applied load between raft bearing pressure and pile resistance. The pile stiffness was typically higher than the raft-soil interface stiffness, resulting in a larger proportion of the pile resistance being activated first. Concurrently, the ensuing small settlement gave rise to small interface pressure below the raft. Arising from complex pile-soil and raft-soil interactions, piles in raft would undergo a higher displacement than a single isolated pile subject to the same pile head load. Conversely, for a given pile head displacement, the single pile and piles in a piled raft would experience different pile head loads.

The experimental data showed that the pile in 16PB3L2 mobilised its full capacity at $\delta = 2.0$ mm, when the applied load $P = 13$ kN. At this load level, the unpiled raft would settle about 5 mm. This was equivalent to a settlement reduction of 60%.

In the creep piling approach, the main function of the piles is to reduce the bearing pressure on the foundation soil. The amount of reduction in bearing pressure at the target settlement must be known if one were to use this approach effectively. It has been suggested that the factor of safety of the piles could be assumed to be 1 in the creep piling approach. Burland proposed that the pile resistance could be estimated

more conservatively by adjusting the sum of the individual pile capacities by a modification factor. In the present study, the effectiveness of pile in moderating raft settlement shall be quantified by a factor η which is not dissimilar to the modification factor proposed by Burland. The meaning of η is illustrated below.

Let δ_a be the allowable settlement and let P_o be the load on the unpiled raft at this allowable settlement. The load required to cause the piled raft to settle by δ_a is designated as P_d (Figure 7.12). The excess load ($P_d - P_o$) carried by the piled raft at the same settlement must be attributed to the piles.

Figure 7.12 Schematics showing the determination of pile contribution factor η

For a piled raft supported by n nos of similar piles with ultimate pile resistance denoted by Q_u , the net contribution factor η of the piles may be defined as:

$$\eta = \frac{P_d - P_o}{n \cdot Q_u} \quad \text{Eqn (7.1)}$$

The η values were determined for all nine model pile rafts at different settlements for applied loads that correspond to different factor of safety (FS) for the unpiled raft. The results are summarised in Table 7.1 and Table 7.3. Figure 7.13 shows the η values plotted against normalised pile spacing. In Figure 7.13, each piled raft is denoted by a symbol with varying sizes and shading. The size and shading of the symbol were used to denote different FS. Thus, the open symbols denote a FS of 7, the shaded symbols denotes a FS of 4 and the solid dark symbol denotes a FS of 3.

As discussed in Chapter 5, the short, narrow pile B2L2 in 4PB2L2 did not enhance the load carrying capacity of piled raft at ultimate state. The η value for 4PB2L2 was less than 0.5 at all levels of FS. The ineffectiveness of pile B2L2 was also observed in 9PB2L2 in which an even smaller value 0.25 was calculated for η .

Figure 7.13 Variation of pile resistance contribution factor (η) with spacing

For the other piled rafts, η appeared to increase with increasing normalised pile spacing, s/B . It was also observed that η would increase as FS decreased. This was to be expected as more of the pile capacities would be mobilised as FS was reduced.

The 16-pile piled raft had very small normalised pile spacing of 2.5 and this resulted in a relatively small η of about 0.6 which did not seem to change much as FS varied from 3 to 7. For FS = 3, the η value for the remaining piled rafts ranged from 0.70 to 1.0. In several cases, η was close to 1.0 signifying full mobilisation of the ultimate pile capacity.

Table 7.1 Contribution factor (η) at $\delta = 1.0$ mm; [$P_o = 4.3$ kN, available factor of safety (FS) against bearing capacity failure for unpiled raft $\cong 7$]

S/ N	Model foundation	No of piles (n)	$\frac{L}{B}$	$\frac{spacing}{B}$	Q_u (kN)	P_d (kN)	$P_d - P_o$ (kN)	$\eta = \frac{(P_d - P_o)}{n.Q_u}$
1	16PB3L2	16	6.7	2.5	0.62	9.69	5.41	0.54
2	9PB3L4	9	13.3	3.3	1.1	10.17	5.89	0.59
3	9PB3L2	9	6.7	3.3	0.62	9.59	5.31	0.95
4	9PB2L4	9	20	5	0.68	8.31	4.02	0.66
5	9PB2L2	9	10	5	0.42	5.06	0.77	0.2
6	4PB3L4	4	13.3	5	1.1	8.51	4.22	0.96
6	4PB3L2	4	6.7	5	0.62	6.3	2.01	0.81
7	4PB2L4	4	20	7.5	0.68	6.96	2.67	0.98
8	4PB2L2	4	10	7.5	0.42	4.97	0.69	0.41

Table 7.2 Contribution factor (η) at $\delta = 2.0$ mm; [$P_o = 7.3$ kN, available factor of safety (FS) against bearing capacity failure for unpiled raft $\cong 4$]

S/ N	Model foundation	No of piles (n)	$\frac{L}{B}$	$\frac{\text{spacing}}{B}$	Q_u (kN)	P_d (kN)	$P_d - P_o$ (kN)	$\eta = \frac{(P_d - P_o)}{n \cdot Q_u}$
1	16PB3L2	16	6.7	2.5	0.62	13.17	5.86	0.59
2	9PB3L4	9	13.3	3.3	1.1	15.31	8	0.81
3	9PB3L2	9	6.7	3.3	0.62	12.84	5.53	0.99
4	9PB2L4	9	20	5	0.68	12.23	4.92	0.8
5	9PB2L2	9	10	5	0.42	10.41	3.09	0.82
6	4PB3L4	4	13.3	5	1.1	11.7	4.39	1
7	4PB2L4	4	20	7.5	0.68	9.98	2.67	0.98
8	4PB3L2	4	6.7	5	0.62	9.36	2.05	0.83
9	4PB2L2	4	10	7.5	0.42	8.1	0.78	0.47

Table 7.3 Contribution factor (η) at $\delta = 3.0$ mm; [$P_o = 9.9$ kN, available factor of safety (FS) against bearing capacity failure for unpiled raft $\cong 3$]

S/ N	Model foundation	No of piles (n)	$\frac{L}{B}$	$\frac{\text{spacing}}{B}$	Q_u (kN)	P_d (kN)	$P_d - P_o$ (kN)	$\eta = \frac{(P_d - P_o)}{n \cdot Q_u}$
1	16PB3L2	16	6.7	2.5	0.62	15.53	5.6	0.56
2	9PB3L4	9	13.3	3.3	1.1	18.65	8.73	0.88
3	9PB3L2	9	6.7	3.3	0.62	15.46	5.53	0.99
4	9PB2L4	9	20	5	0.68	15.17	5.24	0.86
5	9PB2L2	9	10	5	0.42	13.6	3.67	0.97
6	4PB3L4	4	13.3	5	1.1	13.4	3.48	0.79
7	4PB3L2	4	6.7	5	0.62	11.59	1.66	0.67
8	4PB2L4	4	20	7.5	0.68	12.43	2.5	0.92
9	4PB2L2	4	10	7.5	0.42	10.76	0.83	0.49

7.5 Bearing Capacity of Piled Rafts (P_u^*)

To derive maximum benefits and information from the heavy investment of time and effort in the preparation of the model ground and the conduct of each test, the model piled rafts were subjected to repeated load cycles. Although the primary interest was focused on the response in the first load cycle, equal care and attention were exercised in the conduct of the test in subsequent load cycles. The overall multi-cycle load-settlement response of the piled rafts will be discussed in this section.

Similar to the unpiled raft, P - δ curves for these loading cycles exhibited a typical pattern in which the reloading cycles successfully picked out the maximum load applied in the preceding load cycle. The ability to capture the maximum load applied in the preceding load cycle (or ‘memory’ of the loading history) enabled the use of the complete loading envelope in determining the ultimate load bearing capacity of the piled raft. The envelope of the P - δ response in the different load cycles could be taken as representative of a single stage loading from 0 to 50 mm.

The multi-cycle P - δ_{avg} response and the derived loading envelope for 16PB3L2 is shown in Figure 7.14. Similar to the unpiled raft, no obvious ultimate bearing load was observed in the P - δ_{avg} curve for 16PB3L2. Hence, to deduce P_u^* from the P - δ_{avg} curve, the two tangent method and $0.1B_R$ method described in Section 4.2.3 were used. For 16PB3L2, P_u^* obtained using the two tangent method was about 26.5 kN, or 26% higher than the corresponding value for the unpiled raft. The P_u^* obtained using the $0.1B_R$ method was about 30.5 kN. This was nearly the same as that the obtained for the unpiled raft. The equality in ultimate bearing capacities implied that the inclusion of 16 piles in 16PB3L2 did not improve the bearing capacity of the raft. Alternatively, this may be taken to mean that an average raft settlement of $0.1B_R$, the net contribution from the piles to the load carrying capacity of the 16-pile piled raft was negligible. It should be noted that a settlement of $0.1B_R$ occurred only in the third load cycle. This, however, did not mean that the piles were not effective in reducing raft settlement when the applied load was below the ultimate bearing capacity of the raft.

Figure 7.15 shows the P - δ_{avg} curves in the three load cycles for both the unpiled raft and 16PB3L2. In the first and second cycles, at the same average raft settlement, the piled-raft carried a much higher load than the unpiled raft. In other words, the settlement experienced by the piled raft under a given applied load was significantly smaller than that obtained for the unpiled raft. In the 3rd load cycle, the curves for the piled-raft and unpiled rafts converged. At $\delta_{\text{avg}} = 40$ mm, the loads carried by the piled and unpiled rafts were identical. The data corroborated an earlier observation that, at large δ_{avg} when the soil below the raft had yielded, the short piles which were encompassed within the failure soil block had little influence on the load-settlement characteristics of the piled raft.

Figure 7.14 The applied load (P) versus average settlement (δ_{avg}) response of 16PB3L2 in three load cycles and the representative single stage response

Figure 7.15 The applied load (P) versus average settlement (δ_{avg}) response of 16PB3L2 and unpiled raft in three load cycles

Poulos (2000) suggested that the ultimate bearing capacity of the piled raft may be estimated as the smaller of the following two values:

1. Sum of the ultimate capacities of the raft plus all the piles

$$P_u^* = P_u + Q_u^* \quad \text{Eqn (7.2)}$$

Where:

$$Q_u^* = \xi \cdot (n \cdot Q_u)$$

ξ = group efficiency factor

Q_u = bearing capacity of single individual pile

n = number of piles

2. Ultimate capacity of a block encompassing the piles and raft, plus that of the raft portion outside the periphery of the piles

Based on the individual pile capacity determined in the single pile load test, the P_u^* for the 16PB3L2 may be calculated using the above approach as follow:

(1) According to Eqn 7.2,

$$P_u = 26.7 \text{ kN} \{ \text{based on Eqn 4.1, and Terzaghi's bearing capacity factors,} \\ \text{(see Table 4.6)} \}$$

$$Q_u^* = \xi(n \cdot Q_u); \quad \xi = 0.88 \text{ (Poulos \& Davis (1980))}$$

$$Q_u^* = 0.88 \times 16 \times 0.62 = 8.72 \text{ kN}$$

$$P_u^* = 26.7 + 8.72 = 35.4 \text{ kN}$$

(2) Ultimate capacity of a block containing the piles and raft, plus that of the raft portion outside the periphery of the piles

$$P_u^* = 32.50 \text{ kN}$$

The P_u^* deduced from the P - δ_{avg} curves using the two tangent method and the $0.1B_R$ method was about 26.5 kN and 30.5 kN, respectively. The computed ultimate capacity assuming block failure is close to the value obtained $0.1B_R$ method.

Due to the complex interactions involved, Eqn 7.2 was modified by Liu *et al.* (1994) and Borel (2001) to give the following equation.

$$P_u^* = \alpha_{UR} \cdot P_u + \alpha_G \cdot Q_u^* \quad \text{Eqn (7.3)}$$

Where:

α_{UR} and α_G are the modifying coefficients for the failure load of the raft and the pile group when combined in a piled raft

$$\alpha_G = \frac{Q_g}{Q_u^*} \quad \text{Eqn (7.4)}$$

Where:

Q_g = Load carried by piles in the piled raft

$$\alpha_{UR} = \frac{P_u^* - Q_g}{P_u} \quad \text{Eqn (7.5)}$$

Using the deduced value of P_u^* from the P - δ_{avg} curve and the sum of pile head loads Q_g at $P = P_u^*$, the α_{UR} and α_G were back-calculated. In calculating α_G , the pile group efficiency was assumed as unity. The back-calculated α_G was approximately equal

to one. The back-calculated α_{UR} based on P_u^* obtained using the tangent intersection method and the $0.1B_R$ method was about 0.73 and 0.70 respectively.

Based on the numerical investigation conducted on soft clay soil, de Sanctis and Mandolini (2006) proposed Eqn (7.6) for α_{UR} .

$$\alpha_{UR} = 1 - 3[(A_G/A_R)/(s/d)] \quad \text{Eqn (7.6)}$$

Where,

A_G = area enclosed by peripheral piles; A_R = area of the raft

s = spacing between the piles; d = diameter of the piles

Based on Eqn (7.6), the α_{UR} for the 16PB3L2 was about 0.24. The back-calculated α_{UR} from the nine model piled raft tests based on the P_u^* deduced using the two tangent method and the $0.10B_R$ method was about 0.73 and 0.70, respectively. The test results showed that Eqn (7.6) severely underestimated the value of α_{UR} .

The P - δ_{avg} curves and the constructed envelope for the remaining piled rafts are presented in Figures B-1 to B-8 in Appendix B. The deduced values of P_u^* based on the two tangent method and the $0.1B_R$ method and the back-calculated values of α_{UR} and α_G are summarised in Table 7.4.

The bearing capacity of all the piled rafts except 4PB2L2 deduced from the two tangent method was higher than the bearing capacity of the unpiled raft. The P_u^* of 4PB2L2 was about 5% smaller than that obtained for the unpiled raft. Based on the results presented in Table 7.4, the piles improved the bearing capacity of the unpiled raft by 2% to 48%. The back-calculated α_G values assuming group efficiency of unity were about 1.0 to 1.1. The back-calculated α_{UR} values for the nine piled rafts ranged from 0.73 to 1.14. A value of α_G and α_{UR} greater than 1 implied that the assumption of unity for the group efficiency might not be valid.

The P_u^* values deduced from the $0.1B_R$ method revealed a completely different picture. The P_u^* of 4PB2L2, 4PB3L2, 4PB2L4 and 9PB2L2 was smaller than that of unpiled raft. The P_u^* of 9PB3L2 was approximately equal to the unpiled raft capacity. In 4PB3L4 and 16PB3L2 only a marginal increase in P_u^* was observed.

The P_u^* for 9PB2L4 and 9PB3L4 was higher than the unpiled raft capacity by about 16% and 13%, respectively.

The back-calculated α_{UR} values obtained in this study and the values calculated using Eqn (7.6) are summarised in Table 7.5. The α_{UR} for the 4-pile piled rafts obtained using the two tangent method are in reasonable agreement with those calculated using Eqn (7.6). However, for 9-pile piled rafts and 16-pile piled rafts, the experimental α_{UR} were considerably higher than those calculated using Eqn (7.6).

Table 7.4 Bearing capacity of piled rafts and back calculated α_G and α_{UR} values

1	2	3	4	5	6	7	8	9	10	11	12	13	14
	No of piles (<i>n</i>)	<i>Q_u</i> (kN)	<i>n.Q_u</i> (kN)	Two tangent method				0.1 <i>B_R</i> method					
				<i>P_u[*]</i> (kN)	<i>Q_g</i> (kN)	<i>α_G</i>	<i>(P_u[*] - <i>Q_g</i>)</i> (kN)	<i>α_{UR}</i>	<i>P_u[*]</i> (kN)	<i>Q_g</i> (kN)	<i>α_G</i>	<i>(P_u[*] - <i>Q_g</i>)</i> (kN)	<i>α_{UR}</i>
Unpiled raft				21				30					
4PB2L2	4	0.42	1.68	20.0	1.9	1.1	18.1	0.86	24.0	2.0	1.2	22.0	0.73
4PB3L2	4	0.62	2.48	21.5	2.7	1.1	18.8	0.90	26.0	2.8	1.1	23.2	0.77
4PB2L4	4	0.68	2.72	22.0	3.0	1.1	19.0	0.90	29.0	2.7	1.0	26.3	0.88
4PB3L4	4	1.10	4.4	23.8	4.2	1.0	19.6	0.93	30.5	4.7	1.1	25.8	0.86
9PB2L2	9	0.42	3.78	25.0	4.3	1.1	20.7	0.99	27.5	4.5	1.2	23.0	0.77
9PB3L2	9	0.62	5.58	25.0	6.5	1.1	18.5	0.88	30.0	6.6	1.2	23.4	0.78
9PB2L4	9	0.68	6.12	31.0	7.0	1.1	24.0	1.14	35.0	6.7	1.1	28.3	0.94
9PB3L4	9	1.10	9.9	26.0	9.8	1.0	16.2	0.77	34.0	9.0	0.9	25.0	0.83
16PB3L2	16	0.62	9.92	25.0	9.7	1.0	15.3	0.73	30.5	9.5	1.0	21.0	0.70

Table 7.5 The α_{UR} values from the present study and based on the equation proposed by de Sancties & Madolini (2006)

	Spacing, s (mm)	B (mm)	Equivalent, d^* (mm)	A_G (mm ²)	A_R (mm ²)	A_G/A_R	s/d	α_{UR}		
								de Sancties & Madolini (2006)	Two tangent method	0.1 B_R method
4PB2L2	150	20	22.6	2.25x10 ⁴	9x10 ⁴	0.25	6.65	0.89	0.86	0.73
4PB3L2	150	30	33.9	2.25x10 ⁴	9x10 ⁴	0.25	4.43	0.83	0.90	0.77
4PB2L4	150	20	22.6	2.25x10 ⁴	9x10 ⁴	0.25	6.65	0.89	0.90	0.88
4PB3L4	150	30	33.9	2.25x10 ⁴	9x10 ⁴	0.25	4.43	0.83	0.93	0.86
9PB2L2	100	20	22.6	4.0x10 ⁴	9x10 ⁴	0.44	4.43	0.70	0.99	0.77
9PB3L2	100	30	33.9	4.0x10 ⁴	9x10 ⁴	0.44	2.95	0.55	0.88	0.78
9PB2L4	100	20	22.6	4.0x10 ⁴	9x10 ⁴	0.44	4.43	0.70	1.14	0.94
9PB3L4	100	30	33.9	4.0x10 ⁴	9x10 ⁴	0.44	2.95	0.55	0.77	0.83
16PB3L2	75	30	33.9	5.06x10 ⁴	9x10 ⁴	0.56	2.22	0.24	0.73	0.70

7.6 Axial Force Variation along Pile Shaft

In this section the axial force variation along the pile shaft deduced from strain gauge readings is examined. The test data for 4- and 9-pile piled rafts with piles B3L2 and B2L4 are presented. For the remaining model piled rafts reliable data could not be obtained from the strain gauges placed either at mid-level or pile toe. The variation of axial force at two specific pile head settlements (at 0.5 mm and 2.0 mm) i.e. one before the mobilisation of pile capacities and one after the mobilisation of pile capacities is examined.

Figure 7.16 shows the axial force variation of B3L2 pile tested in isolation and when incorporated in the 4- and 9-pile piled rafts at the same pile head settlement of 0.5 mm. The data clearly show that the mobilised end bearing resistance of the piles in the 4- and 9-pile piled raft was not significantly different from that of isolated single pile.

Figure 7.16 Variation of axial force along the pile shaft at a pile head settlement of 0.5 mm for pile B3L2 ($B = 30$ mm & $L = 200$ mm) in single individual case, 4- and 9-pile piled rafts

The shaft resistance mobilised in the upper half of the pile in the 4- and 9-pile piled raft was smaller than that of isolated single pile. The shaft resistance mobilised in the lower half of the pile in the 4- and 9-pile piled raft was not significantly different compared to that of single individual pile. As a consequence, the pile head load of piles in 4- and 9-pile piled raft was smaller than that of isolated single pile.

As discussed in section 7.3, the reason for the smaller pile head load for piles in piled raft were due to the pile-pile and pile-raft interactions. The downward movement of the raft constrained the soil below the raft to be displaced downward. In contrast, the soil in the isolated single was not subjected to the same constraint. The smaller relative pile-soil displacement at the pile shaft resulted in smaller skin friction. This was particularly significant near pile head.

The mobilised average unit skin friction (f_s) along the upper and lower halves for single individual pile and piles in the 4- and 9-pile piled rafts is summarised in Table 7.6. The results clearly shows that while the f_s in single isolated stayed relatively constant throughout the pile length, the piles in the piled raft showed significant difference in the f_s mobilised in the upper and lower halves. There were also noticeable differences in the mobilised f_s along the upper half of the pile due to pile location within the raft. Along the upper half of the pile, the corner pile mobilised higher f_s compared to the centre and edge piles in 9PB3L2.

Table 7.6 Mobilised average unit skin friction, f_s , (kPa) along pile shaft for B3L2 piles tested in isolation and in the 4- and 9-pile piled rafts at different pile head settlements

	At a piled head settlement of 0.5 mm					At a piled head settlement of 2.0 mm			
	Single pile	4PB3L2	9PB3L2			Single pile	4PB3L2	9PB3L2	
			1	2	3			2	3
Upper half	18.3	5.2	2.5	2.5	10.8	18.3	8.3	6.7	10.8
Lower half	18.3	22.5	19.0	18.3	19.2	19.2	26.3	25.8	28.3

At 2.0 mm pile head settlement, the mobilised average skin friction (Table 7.6) along the upper half of the piles in 4- and 9-pile piles raft was significantly smaller than that of single individual pile. The situation was reversed in the lower half of the pile length, with the piles in piled raft registering f_s which were about 40% larger than f_s in single isolated pile.

Figure 7.17 shows the axial force variation of B3L2 at pile head settlement of 2 mm tested in isolation and when incorporated in the 4- and 9-pile piled raft. The data for the centre pile at mid-level and toe was not available. As shown in Figure 7.7, it can be noted that all piles mobilised their ultimate pile head loads when the settlement was 2 mm.

Figure 7.17 Variation of axial force along the pile shaft at a pile head settlement of 2.0 mm for pile B3L2 ($B = 30$ mm & $L = 200$ mm) in single individual case, 4- and 9-pile piled rafts

Figure 7.17 also shows that, pile head loads in 4- and 9-pile piled raft were not significantly different from that of single isolated pile. However, along the upper half the shaft resistance mobilised by piles in piled raft was still smaller than that of single isolated pile.

For model piles with $B=20$ mm and $L=400$ mm

Figure 7.18 shows the axial force variation along the pile shaft at a pile head settlement of 0.5 mm for pile B2L4 tested in isolation and when included in the 4- and 9-pile piled raft. An additional strain gauge was installed at 75 mm below the pile head to monitor the axial force in the piled raft tests. The centre and edge piles in the 9-pile piled raft showed similar q_b as that of isolated single pile, whereas the piles in the 4-pile piled and the corner pile in the 9-pile piled raft showed slightly higher end bearing resistance.

Figure 7.18 Variation of axial force along the pile shaft at a pile head settlement of 0.5 mm for pile B2L4 ($B = 20$ mm & $L = 400$ mm) in single individual case, 4- and 9-pile piled rafts

The shear resistance mobilised by piles in the two piled rafts along the lower half was similar to that of single individual pile except for the centre pile in the 9-pile piled raft. The shear resistance along the upper half of the pile was significantly smaller than that of single individual pile. There was very little skin friction in the upper $1/5^{\text{th}}$ of the piles in the piled raft. This is more clearly seen in the results for pile B2L4 shown in Figure 7.19.

At a pile head settlement of 2 mm, the shaft resistance mobilised in the lower half of piles in both the 4- and 9-pile piled raft was slightly higher than that observed in the single isolated pile. In contrast, f_s along the upper half portion of piles in the piled rafts was considerably smaller. More interestingly, the axial load distribution profile and average unit skin friction presented in Table 7.7 showed the existence of negative skin friction in the upper $1/5^{\text{th}}$ of the piles. The close clustering of the axial load profiles gave confidence that the experimental data are reliable.

Figure 7.19 Variation of axial force along the pile shaft at a pile head settlement of 2.0 mm for pile B2L4 ($B = 20$ mm & $L = 400$ mm) in single individual case, 4- and 9-pile piled rafts

Table 7.7 Mobilised average unit skin friction, f_s , (kPa) along pile shaft for B2L4 piles tested in isolation and in the 4- and 9-pile piled rafts at different pile head settlements

Depth below pile head (mm)	At a piled head settlement of 0.5 mm					At a piled head settlement of 2.0 mm				
	Single pile	4PB2L4	9PB2L4			Single pile	4PB2L4	9PB2L4		
			1	2	3			1	2	3
0-75	18.0	-0.9	-3.0	-1.4	8.9	18.8	-3.3	-6.6	-4.7	5.0
75-200		13.2	14.0	11.1	11.0		15.5	18.4	13.9	15.8
200-400	18.5	16.2	12.7	16.3	13.5	19.3	24.7	25.2	27.1	24.7

Observation

At different pile head settlements, the end bearing resistance mobilised by piles in 4- and 9-pile piled rafts was not significantly different from that of isolated single pile. When the pile capacity was fully mobilised, the average unit skin friction along the bottom half portion of the pile was similar to that of single individual pile. Along the upper 1/5th of the pile, at smaller settlement the f_s was either small or zero. With the increase of settlement, f_s became negative indicating the impact of relative pile-soil displacement just beneath the raft.

7.7 Summary

The development of interface contact pressure and pile head load with raft displacement are examined in this chapter. The data on interface contact pressure were recorded by the total stress transducers installed on the underside of the raft. The strain gauges on selected piles provided information on pile head load.

The test data showed that the contact pressure-displacement (σ_c - δ) relationship or soil spring curve at the raft-soil interface was highly non-linear, even at small raft displacement. The soil spring curve varied from location to location. The curve at the corner was found to be stiffer than the soil spring curve at the quarter points. However, at moderate stress level, the curve at a given location appeared to be unique and was independent of pile dimension and the number of piles present in the raft. For contact stress level below 60 kPa, the soil spring curves at the corner and quarter point differed only slightly. Hence, the assumption a unique non-linear

soil spring curve might suffice when modelling the piled raft response in low to moderate stress level.

Pile Head Load versus Piled Head Displacement Response

Due to complex pile-soil-pile and pile-raft interaction, the pile stiffness (pile head load to displacement ratio) within the raft would be smaller than the stiffness of a single pile. The model tests showed that the change in pile stiffness was affected significantly by the number of piles present. Hence, it was also influenced by the normalised pile spacing since the raft size was kept constant in all the tests. The changes in stiffness were particularly pronounced for short piles. Within the bounds of experimental error, the ultimate capacity of the piles in the raft did not appear to be affected by pile-soil interactions.

Mobilisation of Pile Capacity at Target Raft Settlement

The contribution of pile resistance at target raft settlement was quantified in terms of the factor η which was a measure of the fraction of ultimate pile capacity mobilised. It was found that when piles were spaced far apart, the full pile capacity could be effectively mobilised (i.e. $\eta = 1$). It was also found that η values for short piles and closely spaced piles could be lower than 0.5. Generally, a value of 0.75 might be assumed for piles at normalised pile spacing greater than 4.

To optimise the utilisation of the pile capacity for the purpose of reducing raft settlement, it is preferable to use a small number of stiff piles of high capacities, spaced far apart, than a large number of short piles of lower capacity.

Bearing Capacity of Piled Raft (P_u^)*

The ultimate bearing capacities of the piled raft were interpreted from the envelope encompassing the multi-cycle load-settlement curves. The bearing capacity deduced using the two interpretation methods tended to give different P_u^* values. For rafts with short piles, the bearing capacity was not different from that of the unpiled raft. The biggest increase in capacity over the unpiled raft was 48%. The back-calculated α_G assuming group efficiency of unity varied from 1.0 to 1.1. The back-calculated α_{UR} varied from 0.73 to 1.14, much higher than the values.

Chapter 8

Three Dimensional Finite Element Analyses of Unpiled Raft and Single Individual Piles

8.1 Introduction

Extensive experimental work has been carried out to study the load-settlement response of raft supported on piles. Extreme care was exercised in the soil specimen preparation to ensure uniform and consistent specimen properties. However, as highlighted in the previous chapters, even with proper care, relatively small variations were inevitable in the properties of the various soil specimens. Most importantly the time and effort required to carry out a detailed parametric study through experimental work are substantial. Moreover, with the adopted instrumentation the overall contact pressure distribution, the load distribution along the pile shaft could not be explored in-depth. With the help of appropriately calibrated numerical model the parametric studies can be extended considerably and the behaviour of raft supported on piles could be studied more comprehensively. Besides this, the comparison of experimental results with numerical results also gives a scope for assessing the adequacy of the numerical model.

The interaction of the piles, raft and soil in a piled raft foundation is an extremely complex problem. Previous attempts in rigorous analyses (e.g. Butterfield & Banerjee, 1971; Desai, 1974; Hooper, 1974; Brown & Wiesner, 1975; Hain & Lee, 1978; Kuwabara, 1989; Franke *et al.*, 1994; Ta & Small, 1996; Sinha, 1997; Russo & Viggiani, 1998; Zhang & Small, 2000) inevitably involve simplification with regards to the modelling of the soil and interface response. Mindlin's solution of a point load in an elastic half space formed the basis of the most pile group analysis (Poulos & Davis, 1980). Though these highly idealised solutions provide some insight into the pile group response, they do not provide a sufficient basis for a thorough understanding of the load transfer mechanism. In this study, three

dimensional non-linear analyses of the piled rafts using the finite element code Aabaqus version 6.4 were carried out. In the following sections, the modelling and results obtained are presented.

The finite element model was first calibrated using the data obtained from model tests on unpiled raft and single individual piles. The calibrated finite element model was then used to study the behaviour of the piled rafts and results obtained through the numerical analyses were compared against the experimental results.

8.2 Finite Element Modelling

As described in the previous sections, the commercially available finite element software Abaqus version 6.4 was used to carry out the three dimensional analyses. The finite element software Abaqus was developed by Hibbitt, Karlsson & Sorensen, Inc. of USA. This software was very widely used by researchers for the analyses of piled rafts, laterally loaded piles, and other geotechnical problems (Cao, 1998; Reul & Randolph, 2003; Reul & Randolph 2004; Reul, 2004; Katzenbach *et al.*, 2005; Miao, 2005; de Sanctis & Mandolini, 2006). In the following sections, the constitutive models for soil, raft and piles, interaction models and boundary conditions adopted in the present study are discussed.

Soil Material Model

In Abaqus/Standard a wide range of material models are available to model the elasto-plastic behaviour of soil (Mohr-Coulomb, extended Drucker-Prager, modified Drucker-Prager, Cam-Clay etc.). In the present study, based on the loading rate applied in the tests and due to relatively low permeability of the model ground, the condition was neither fully undrained nor fully drained. Partial drainage would most likely have taken place during loading. A more appropriate numerical modelling should adopt a coupled consolidation approach. However, an undrained analysis may be adequate to reveal the broad mechanism of load transfer in piled raft. As the Mohr-Coulomb model is generally considered to be adequate in most practical applications, the undrained behaviour of soil was simulated using Mohr-Coulomb model. However, with the combination of interaction models used, it was found that the Mohr-Coulomb model gave rise to convergence problems for certain

specimens of piled rafts at large displacements. Hence, the linear Drucker-Prager material model was adopted instead. The Drucker-Prager model parameters were chosen such that it would provide the same failure definition in triaxial compression and tension as the Mohr-Coulomb model. In the following sections the linear Drucker-Prager model and comparison of results obtained by using Mohr-Coulomb model and Drucker-Prager model are presented.

The Drucker-Prager model is written in terms of all the three stress invariants. It provides for a possibly noncircular yield surface in the deviatoric plane to match different yield values in triaxial tension and compression, associated inelastic flow in the deviatoric plane, and separate dilation and friction angles.

Stress Invariants

The stress invariants are defined as follows

$$p = -\frac{1}{3}\text{trace}(\sigma) \quad \text{Eqn (8.1)}$$

and the Mises equivalent stress,

$$q = \sqrt{\frac{3}{2}(\mathbf{S}:\mathbf{S})} \quad \text{Eqn (8.2)}$$

where, $\mathbf{S}$ is the stress deviator, defined as

$$\mathbf{S} = \sigma + p\mathbf{I} \quad \text{Eqn (8.3)}$$

In addition, the linear model also uses the third invariant of deviatoric stress,

$$r = \left(\frac{9}{2}\mathbf{S}:\mathbf{S}:\mathbf{S}\right)^{\frac{1}{3}} \quad \text{Eqn (8.4)}$$

Yield Criterion

The linear Drucker-Prager yield criterion (Figure 8.1a) is written as

$$F = t - p \tan \beta - D = 0 \quad \text{Eqn (8.5)}$$

(a) Linear Drucker-Prager yield surface in meridional plane

(b) Typical yield/flow surfaces of the linear model in the deviatoric plane

(c) Mohr-Coulomb model in the deviatoric plane

Figure 8.1 Linear Drucker-Prager model (after Drucker and Prager, 1952)

Where,

$$t = \frac{1}{2}q \left[1 + \frac{1}{K} - \left(1 - \frac{1}{K} \right) \left(\frac{r}{q} \right)^3 \right] \quad \text{Eqn (8.6)}$$

β is the slope of the linear yield surface in the $p-t$ stress plane and d is the intercept of the yield surface.

K is the ratio of the yield stress in triaxial tension to the yield stress in triaxial compression and thus controls the dependence of the yield surface on the value of the intermediate principal stress.

Determination of Drucker-Prager Model Parameters

The Drucker-Prager model parameters were obtained by making the Mohr-Coulomb and Drucker-Prager models yield the same failure definition in triaxial compression and tension.

The Mohr-Coulomb model in terms of principal stresses can be written as follows,

$$\sigma_1 - \sigma_3 + (\sigma_1 + \sigma_3) \sin \phi - 2c \cos \phi = 0 \quad \text{Eqn (8.7)}$$

Using the results above for the stress invariants p , q , and r in triaxial compression and tension allows the linear Drucker-Prager model to be written for triaxial compression as

$$\sigma_1 - \sigma_3 + \frac{\tan \beta}{2 + \frac{1}{3} \tan \beta} (\sigma_1 + \sigma_3) - \frac{1 - \frac{1}{3} \tan \beta}{1 + \frac{1}{6} \tan \beta} \sigma_c^0 = 0 \quad \text{Eqn (8.8)}$$

and for triaxial tension as

$$\sigma_1 - \sigma_3 + \frac{\tan \beta}{\frac{2}{K} + \frac{1}{3} \tan \beta} (\sigma_1 + \sigma_3) - \frac{1 - \frac{1}{3} \tan \beta}{\frac{1}{K} + \frac{1}{6} \tan \beta} \sigma_c^0 = 0 \quad \text{Eqn (8.9)}$$

These expressions were made identical to the Mohr-Coulomb model for all values of (σ_1, σ_3) by setting up

$$K = \frac{1}{1 + \frac{1}{3} \tan \beta} \quad \text{Eqn (8.10)}$$

By comparing the Mohr-Coulomb model with the linear Drucker-Prager model,

$$\tan \beta = \frac{6 \sin \phi}{3 - \sin \phi} \quad \text{Eqn (8.11)}$$

$$\sigma_c^0 = 2c \frac{\cos \phi}{1 - \sin \phi} \quad \text{Eqn (8.12)}$$

and from the previous result

$$K = \frac{3 - \sin \phi}{3 + \sin \phi} \quad \text{Eqn (8.13)}$$

These results for β , K and σ_c^0 provide linear Drucker-Prager parameters that match the Mohr-Coulomb model in triaxial compression and tension.

For undrained case, $\beta = 0$, $K = 1$, and $\sigma_c^0 = 2c_u$,

Raft and Pile Material Model

The raft and piles were modelled as linear elastic material. The model raft used in this study was a composite member of two individual plates made of aluminium. As discussed in Chapter 3, in the bottom plate grooves were made for concealing the instrument wires and the effective stiffness of the raft was reduced as a result of this. Hence, for determining the stiffness of the raft a calibration test was carried out. This was done by applying a central point load on the raft supported on the rollers as shown in Figure 8.2. The load-settlement response was back analysed using finite element modelling. The thickness of the raft was taken as 15 mm and the equivalent Young's modulus of elasticity of the raft (E_r) was obtained by

matching the experimental and numerical load-displacement behaviour. The equivalent E_r value thus obtained was around 2.8×10^7 kPa. The hand book value for aluminium is between 6.5×10^7 to 7.0×10^7 kPa.

Figure 8.2 Schematics of the test set-up for determining equivalent stiffness of the raft

To obtain the representative Young's modulus of elasticity for the model piles (E_p) made of aluminium, an axial compression test was conducted on the model pile using uniaxial compression testing machine. Based on the measured axial stress versus strain response, the E_p value was obtained. The E_p value thus obtained was around 6.85×10^7 kPa, which is within the range specified for aluminium material (6.5×10^7 kPa to 7.0×10^7 kPa). In the present study, the hollow square piles were modelled as square solid piles with the axial stiffness of solid piles reduced to be equivalent to the axial stiffness of hollow square piles. The elastic properties used in the analyses for the raft and piles are summarised in Table 8.1.

Table 8.1 Elastic properties adopted in the numerical analyses for model raft and model piles

	E (kPa)	ν
Raft	2.80×10^7	0.35
Piles, $B=20$ mm	2.47×10^7	0.35
Piles, $B=30$ mm	1.70×10^7	0.35

Element Types

The soil and piles were modelled using solid continuum 8-node brick elements with reduced integration (element type C3D8R). The raft was modelled using 3D shell elements (S8R).

Interaction

With the objective of getting raft-soil contact pressures, the interaction between the raft and soil was modelled using "hard" contact model with tension cut-off. This model allows for transfer of any amount of pressure when both the surfaces are in contact and no pressure is transferred when the surfaces are not in contact (Figure 8.3). The "hard" contact model offers zero resistance for sliding and the raft surface was considered as smooth.

In the experimental investigation, the model raft and piles were rigidly connected to each other. To simulate this condition, the interaction between the raft and piles was modelled using "TIE" contact. The "TIE" contact allows no relative displacement between the raft and piles at the interface.

Figure 8.3 "Hard" contact model pressure-over closure relationship

The interface between the pile shaft to soil was modelled as perfectly rough, no relative motion between the pile shaft and soil element in contact with pile shaft was allowed. In this case to model the interface no special elements were used; instead the thickness of the soil elements at the interface was kept approximately as 0.1 times the width of the piles as adopted by de Sanctis & Mandolini (2006). Around the pile a smeared zone approximately equal to half the width of the pile on each side was created. The strength and stiffness properties were varied for this zone of soil to match the load settlement behaviour of single individual piles observed experimentally. The soil and piles were modelled without consideration of installation effects.

Geometry and Boundary Conditions

Taking advantage of symmetry, only one-quarter of the physical piled raft model was considered in the finite element model. Figure 8.4 shows the typical finite element mesh for model unpiled raft. The size of square domain modelled was equal to the radius of the consolidation tank i.e. 0.5 m, the height of the FE model was 0.7 m which was the height of the model ground in piled raft model test.

The surface ABCD and CDEF shown in Figure 8.4(b) were symmetrical about y-axis and x-axis respectively. Hence symmetrical boundary conditions respectively $U_x = U_y = U_z = 0$ and $U_y = U_x = U_z = 0$ were applied. The boundaries in x-, y- and z-directions were constrained to move in their respective directions. The number of nodes, elements and degrees of freedom were respectively about 8000, 7650 and 26000.

Where:

U_x & U_y = displacements in x & y directions respectively

U_{R_x} , U_{R_y} & U_{R_z} = rotations in x, y & z directions respectively

Output

Central settlement: The nodal displacement at the centre of the raft in z-direction was taken to be the central settlement of the raft.

Average settlement: The average settlement of the raft was calculated by taking the area weighted average of the displacement of all the nodes of the raft in z-direction.

Figure 8.4 Finite element mesh pattern and boundary conditions for unpiled raft

Maximum differential settlement: The true maximum differential settlement in this case was the displacement of node at centre of the raft minus displacement of node at corner of the raft. However, in the experimental investigation the corner settlement was measured approximately at 30 mm diagonally inside. Hence for direct comparison, the settlement at the corresponding location was used in computing the differential settlement.

Axial load carried by piles: The axial load mobilised by pile at different elevations was calculated based on the average of axial stress at integration points of all the elements representing the pile.

Contact pressure: The contact pressure can be obtained directly from Abaqus output. To validate the contact pressure data obtained using "hard" contact model, flexible strip footing and rigid strip footing on elastic soil were analysed and results were compared against available analytical solutions. These results would be presented in the subsequent sections.

Calibration of Finite Element Models

Comparison of results obtained using Mohr-Coulomb model and Drucker-Prager model

The load versus average settlement obtained using the Mohr-Coulomb model and Drucker-Prager model is presented in Figure 8.5. In both cases, for $P < 20$ kN the load versus settlement response obtained using both the models agree closely. For $P > 20$ kN, the predicted δ_c using Mohr-Coulomb model was higher than the δ_c obtained using Drucker-Prager model. The bearing capacity interpreted based on $0.1B_R$ method in the case of Mohr-Coulomb model (extrapolated) was approximately 4% less than the value in the case of Drucker-Prager model.

Figure 8.5 Applied central point load (P) versus average settlement response (δ_{avg}) using Mohr-Coulomb and Drucker-Prager models

Validation of Contact Pressure Obtained Using "Hard" Contact Model

To verify the correctness of the contact pressure data, a vertical equilibrium check was made between the applied load and back calculated load based on the contact pressure data. A weighted average of the contact pressure was calculated assuming linear distribution between the nodes. The load was back calculated using the average contact pressure. Figure 8.6 shows the variation of back calculated load with the applied load and percentage difference between these two values. The back calculated load was higher than the applied load. The percentage difference between the back calculated load and applied load increased with the increase of the applied load. This may be due to the increase in the non-linearity of the contact pressure distribution. However, even at $P=30$ kN the percentage difference was only order of 3% which can be considered as negligible. To further verify the contact pressure distribution obtained using "hard" contact model the case of fully flexible and perfectly rigid strip footing on an elastic soil were analysed.

Figure 8.6 Difference in the back calculated load using contact pressure and applied load

Flexible Raft

To model a perfectly flexible footing, the Young's modulus of the raft was taken as equal to that of soil, and a uniformly distributed load was applied on the footing. The soil and raft properties used are summarised in Table 8.2. The settlement profile and contact pressure are shown in Figure 8.7. As expected, a uniform contact pressure equal to that of applied pressure was mobilised throughout the raft.

Table 8.2 Raft and soil properties

	Raft		Soil	
	E (kPa)	ν	E (kPa)	ν
Flexible strip footing	9.0×10^3	0.35	9.0×10^3	0.35
Rigid strip footing	2.8×10^{12}	0.35	9.0×10^3	0.35

Figure 8.7 Interface contact pressure (σ_c) and settlement of a flexible strip footing

Rigid raft

The properties adopted for modelling the rigid strip footing are presented in Table 8.2. To model a perfectly rigid footing, the better choice is performing a settlement controlled analyses by applying uniform settlement throughout the footing. However, in order to compare the results directly with the theoretical values for perfectly rigid strip footing on elastic soil, a load controlled analyses was performed and a very high Young's modulus of elasticity (2.8×10^{12} kPa) was adopted for the rigid strip footing. The observed settlement at different load levels and the ratio of contact pressure to applied pressure (σ_c/p_a) was presented in Figure 8.8. At any given location, the ratio of σ_c/p_a was independent of applied pressure, p_a . The numerically obtained σ_c/p_a value was the same as the theoretical value for perfectly rigid strip footing on elastic soil except at the corner of the strip footing. At corner of the footing, theoretically the contact pressure was infinite whereas in the numerical analyses a finite value of contact pressure was observed.

Figure 8.8 Interface contact pressure (σ_c) and settlement (δ) of a rigid strip footing

8.3 Back Analysis of Unpiled Raft

The initial tangent modulus (E_u) obtained in the UU tests on the soil specimens sampled after model tests varied from 7500 kPa to 8800 kPa. The c_u value was between 38.5 kPa to 42 kPa. Hence, to calibrate the load-settlement behaviour of UR40 using the numerical model, five different values of E_u were used and the results were compared with the experimental results. An undrained shear strength $c_u = 40$ kPa was adopted in the FE Analyses. Instead of cyclic loading, monotonic loading was adopted in the FE analyses.

Central Settlement

The computed applied load (P) versus central settlement (δ_c) response using different values of E_u is presented in Figures 8.12. For comparison, the results from model unpiled raft test (UR40) are superimposed in these figures.

As described in Chapter 4, in the P - δ_c response obtained from model test UR40, the change in the gradient was relatively small for $P < 10$ kN. The initial gradient for the P - δ_c at zero load (k_o) was approximately 3.1 kN/mm.

Figure 8.9 clearly indicates that, when $E_u = 6000$ kPa was adopted in the FE analysis, the predicted P - δ_c response was similar to that observed experimentally up to $P = 10$ kN. However for $P > 11$ kN, the predicted P - δ_c response was stiffer than the experimental data. Thus, for $11 \text{ kN} < P < 28 \text{ kN}$, the predicted δ_c was at any given load was smaller than the measured δ_c .

Figure 8.9 Applied central point load (P) versus predicted central settlement (δ_c) adopting different values of Young's modulus for soil

Based on the computed P - δ_{avg} curve for $E_u = 6000$ kPa, the bearing capacity deduced using the tangent intersection methods and the $0.1B_R$ method was 24 kN and 29.5 kN, respectively. The corresponding value obtained from the model test was 21 kN and 30 kN, respectively.

In summary, by using simple linear elasto-plastic material model for soil and by adopting $E_u = 6000$ kPa, the P - δ_c response was closely depicted up to $P = 10$ kN. The bearing capacity deduced from the computed P - δ_{avg} was in reasonable agreement with the experimental value. However, for $11 \text{ kN} < P < 28 \text{ kN}$, the computed settlement was much smaller than experimental value.

As expected, the computed initial gradient of P - δ_c curve increases with increasing E_u . The initial gradient of P - δ_c response was increased by about 48% as E_u increases from 6000 to 10000 kPa. However, the bearing capacity was increased only by about 8%. The computed initial gradient of P - δ_c curve and the computed bearing capacity with varying E_u values are summarised in Table 8.3.

Table 8.3 Summary of initial tangential stiffness and P_u for different cases of unpiled raft

	initial gradient of P - δ_c response, k_o (kN/mm)	P_u (kN) (using $0.1B_R$ method)
UR40 (exp-virgin loading)	3.1	30.0
UR40 (exp-reloading)	4.1	
E_u : 6000 kPa	3.1	29.5
E_u : 7000 kPa	3.7	30.2
E_u : 8000 kPa	4.0	30.8
E_u : 9000 kPa	4.2	31.4
E_u : 10000 kPa	4.6	31.9

Figure 8.10 Applied central point load (P) versus predicted differential settlement ($\Delta\delta$) adopting different values of Young's modulus for soil

Differential settlement

The applied load (P) versus differential settlement ($\Delta\delta$) responses obtained for different E_u is presented in Figure 8.10. For comparison the results from model tests are also presented in the same figure. It was found that E_u has negligible effect on $\Delta\delta$. The predicted P - $\Delta\delta$ response was in close agreement with the response obtained in the model test.

Contact pressure versus settlement

The measured contact pressure versus settlement response at corner and quarter points and the corresponding predicted response from FE analyses are presented in Figure 8.11 and 8.12 respectively. It is obvious that a change in E_s from 6000 kPa and 9000 kPa does not lead to significant changes in σ_c - δ curves.

At corners, the computed curve only agrees with the measure response when the contact stress is low. For σ_c exceeding 100 kPa, there was large divergence in the computed and measure response, except when at $\delta=16$ mm where the two curves coincide. The single stage FE calculation yielded an ultimate contact stress of about 240 kPa, which was much smaller than the contact pressure obtained in the third load cycle of the model test.

Similar observations could also be made about the contact pressure – displacement relationship at the quarter points. Apart from the similarity at the initial stage of loading, large discrepancies between the computed and experimental curves were observed.

Figure 8.11 Interface contact pressure (σ_c) versus settlement (δ) response at corners obtained from FE analyses and model tests

Figure 8.12 Interface contact pressure (σ_c) versus settlement (δ) response at quarter points obtained from FE analyses and model tests

The variation of interface contact pressure with average applied pressure

The variation of interface contact pressure (σ_c) at corners and quarter points with average applied pressure (p_a) is presented in Figure 8.13 and 8.14. The predicted σ_c at corners showed reasonable agreement with the measured σ_c in model test up to p_a of 140 kPa. For $p_a > 140$ kPa, smaller σ_c was predicted in the FE analysis. The computed σ_c at quarter points (Figure 8.14) was slightly smaller than the measured σ_c for $p_a < 100$ kPa, but exceed the experimental value for $p_a > 120$ kPa.

Figure 8.13 Mobilisation of interface contact pressure (σ_c) with applied load (P) at corners for unpiled raft obtained from FE analyses and model test

Figure 8.14 Mobilisation of interface contact pressure (σ_c) with applied load (P) at corners for unpiled raft obtained from FE analyses and model test

8.4 Back Analysis of the Single Piles

As discussed in Section 8.2, the interface between the pile shaft and soil was assumed to be perfectly rough and a smeared zone approximately equal to half of its size on each side was created around the pile as shown in Figure 8.15. The shear strength of the soil in this zone was given as αc_u (c_a). The thickness of the soil elements at the interface was kept equal to 0.1 times the width of the piles.

The c_u and E_u obtained from the UU tests on the soil specimens retrieved after model tests varied from 38.5 kPa to 42.0 kPa and 7500 kPa to 8800 kPa, respectively (Table 3.7). As discussed in Chapter 4, the back-calculated α value single pile load test varied from 0.48 to 0.51 for the 4 model piles (Table 4.11). A series of calibration exercises adopting different values of E_u , c_u and α values for the smeared zone were carried out to derive a suitable set of soil parameters for the FE analysis. The properties for soil region outside smeared zone were kept constant.

The c_u , E_u and αc_u (c_a) values adopted in the first trial were summarised in Table 8.4. In the first trial, the E_u for the smeared zone was taken to be the same as the soil outside the smeared zone i.e. 8800 kPa.

The computed and measured pile head load versus pile head displacement curves of the four model piles are presented in Figures 8.16 to 8.19. The initial tangential stiffness (k_p) and the ultimate resistance of the pile (Q_u , determined by tangent intersection method) obtained from model tests and FE analyses are summarised in Table 8.5. For pile B3L4 (breadth=30 mm and length=400 mm), the computed k_p was similar to the experimental value. For the other three piles the computed k_p was around 9% to 18% higher than the experimental values. The computed Q_u for different piles was 25% to 48% higher than test values. Since the computed k_p and Q_u were higher than the experimental values, the E_u and c_u for the smeared zone were adjusted downwards in subsequent analyses to reduce the discrepancy between the computed and measured curves.

Figure 8.15 Finite element mesh pattern for single piles

Table 8.4 Material properties adopted in the FE analyses for single isolated piles

	E (kPa)	ν	
Piles, $B=20$ mm	2.47×10^7	0.35	-
Piles, $B=30$ mm	1.70×10^7	0.35	-
Adopted soil properties in first trial			
Soil out side the smeared zone	8800	0.49	$c_u = 40.0$ kPa; $\phi_u = 0^\circ$
Soil within the smeared zone	8800	0.49	$c_a = 19.2$ kPa ($\alpha=0.48$); $\phi_u = 0^\circ$
Adopted soil properties in final trial			
Soil out side the smeared zone	8800	0.49	$c_u = 40.0$ kPa; $\phi_u = 0^\circ$
Soil within the smeared zone	8000	0.49	$c_a = 16.0$ kPa ($\alpha=0.40$); $\phi_u = 0^\circ$

Figure 8.16 The computed and measured pile head load versus pile head displacement response of isolated single pile B2L2 ($B=20$ mm & $L=200$ mm)

Figure 8.17 The computed and measured pile head load versus pile head displacement response of isolated single pile B3L2 ($B=30$ mm & $L=200$ mm)

Figure 8.18 The computed and measured pile head load versus pile head displacement response of isolated single pile B2L4 ($B=20$ mm & $L=400$ mm)

Figure 8.19 The computed and measured pile head load versus pile head displacement response of isolated single pile B3L4 ($B=30$ mm & $L=400$ mm)

Table 8.5 Initial tangential stiffness (k_p) and Q_u for different individual piles from FE analyses and model tests using original soil parameters

Specimen	Initial tangential stiffness, k_p (kN/mm)		Q_u (kN)	
	Exp	Num	Exp	Num
SPB20L200	1.02	1.20	0.42	0.58
SPB30L200	1.38	1.50	0.62	0.92
SPB20L400	1.31	1.50	0.68	0.85
SPB30L400	2.00	2.00	1.10	1.40

The overall ‘best’ match between the computed and experimental curves was obtained for $E_u = 8000$ kPa and $c_a = 16.0$ kPa in the smeared zone. The properties adopted in the final trial are summarised in Table 8.4.

The pile head load versus displacement response obtained from FE analyses using these E_u and c_a values are shown Figures 8.16 to 8.19. The computed initial tangential stiffness (k_p) and Q_u obtained are summarised in Table 8.6. The maximum difference in the computed and measured k_p was around 10%; the difference between the computed and measured Q_u was less than 13%.

Table 8.6 Initial tangential stiffness (k_p) and Q_u for different individual piles from FE analyses and model tests using modified soil parameters

Specimen	Initial tangential stiffness, k_p (kN/mm)		Q_u (kN)	
	Exp	Num	Exp	Num
SPB20L200	1.02	1.00	0.42	0.45
SPB30L200	1.38	1.30	0.62	0.70
SPB20L400	1.31	1.35	0.68	0.75
SPB30L400	2.00	1.80	1.10	1.18

The total pile head load can be decomposed into shaft resistance (Q_s) and end bearing resistance (Q_b). Figures 8.20 to 8.23 show the computed variation of Q_s and Q_b with pile head displacement. It was observed that in both the experiment and FE analyses, Q_s was fully mobilised at pile head displacement of less than 1 mm. The measured data showed that after reaching a peak value, Q_s tended to decrease to a final ultimate value. No peaks were observed in the FE computations, Q_s was found to tapered smoothly to the ultimate values.

Both tests and FE simulations showed the full mobilisation of the ultimate end bearing pressure Q_b did not occurred when the pile head displacement had reached 5 mm. Unlike the shaft resistance, the Q_b curve increased monotonically to the ultimate value.

Figure 8.20 The computed and measured shaft and end bearing resistance versus pile head displacement response of isolated single pile B2L2 ($B=20$ mm & $L=200$ mm)

Figure 8.21 The computed and measured shaft and end bearing resistance versus pile head displacement response of isolated single pile B3L2 ($B=30$ mm & $L=200$ mm)

Figure 8.22 The computed and measured shaft and end bearing resistance versus pile head displacement response of isolated single pile B2L4 ($B=20$ mm & $L=400$ mm)

Figure 8.23 The computed and measured shaft and end bearing resistance versus pile head displacement response of isolated single pile B3L4 ($B=30$ mm & $L=400$ mm)

8.5 Summary

The finite element software ABAQUS 6.4 was used to model the unpiled rafts single piles. Undrained condition was assumed to prevail and a total stress analysis was conducted. The combination of Mohr-coulomb model for soil and the adopted interface model resulted in convergence problems. Hence, the Drucker-Prager linear elasto-plastic material model was adopted.

Unpiled raft

A series of calibration studies were carried out using the unpiled raft to determine the soil parameters for the FE analysis. In the calibration exercise, c_u was fixed at 40 kPa and E_u was allowed to vary within the range of values obtained from UU tests. Based on the calibration exercise, E_u was fixed at 6000 kPa.

Although a good match was obtained for the computed and measured load-displacement curves for the unpiled raft, the measured contact pressure response had not been successfully reproduced in the FE calculations. The discrepancy was particularly pronounced at large displacement.

Isolated single piles

When the soil parameters derived from triaxial tests were used in the FE analysis, the computed initial stiffness was 9%-18% higher than the measured values. The computed capacity was higher than the measured values by 25% - 48%. By a series of calibration exercise in which the soil parameters were adjusted, the best match between the computed and measured load-settlement curves was achieved when E_u in the smeared zone was reduced by 12.5% and c_a was reduced by 17% from the experimentally derived values.

Chapter 9

Three Dimensional Finite Element Analysis of Piled Rafts – and Comparison with Model Tests Results

9.1 Introduction

In order to gain a deeper insight into the mechanism of settlement reduction by piles in a raft, the piled raft tests described in preceding chapters were modelled numerically in a series of 3-D finite element simulations using Abaqus. The soil properties used in the finite element analyses were derived from calibration exercise on unpiled raft and single pile tests described in Chapter 8. The properties of the model piles and the raft were similarly derived from calibration exercise validated against finite element simulations described in Chapter 8. The parameters are summarised in Table 9.1.

The overall response obtained from finite element (FE) analysis for one typical model piled raft (9PB3L2) was analysed in detail and compared with the experimental data. Subsequently, the finite element output would be compared with the major observations obtained from the nine model piled rafts.

9.2 Finite Element Analyses of Model 9-pile piled Raft (9PB3L2)

The 9-pile piled raft comprised piles 30 mm wide (B) and 200 mm long (L). The centre to centre spacing between the piles was $3.3B$. The material properties summarised in Table 9.1 were used in the finite element analyses. In the following sections the finite element model, the predicted response using finite element modelling viz. load-settlement response and load transfer through piles and raft-soil interface would be verified against experimental data.

Table 9.1 Material properties adopted in the numerical analyses

	E (kPa)	ν	
Soil within the smeared zone	8000	0.49	$c_a = 16$ kPa; $\phi_u = 0^\circ$
Soil out side the smeared zone	9000	0.49	$c_u = 40$ kPa; $\phi_u = 0^\circ$
Raft	2.80×10^7	0.35	-
Piles, $B = 20$ mm	2.47×10^7	0.35	-
Piles, $B = 30$ mm	1.70×10^7	0.35	-

Finite element model

Figure 9.1 shows the typical finite element mesh for model piled raft 9PB3L2. By taking advantage of symmetry, only one-quarter of the piled raft was modelled in the finite element analysis. The size of the square domain modelled was set equal to the radius of the consolidation tank i.e. 0.5 m, the height of the FE model at 0.7 m was equal to the height of the model ground used in the piled raft model tests.

The surface ABCD and CDEF shown in Figure 9.1(b) were symmetrical about the y-axis and the x-axis, respectively. Hence symmetrical boundary conditions $U_x = U_y = U_z = 0$ and $U_y = U_x = U_z = 0$ were applied. The boundaries in the x-, y- and z-directions were constrained to move in their respective directions.

As described in Chapter 8, the soil was modelled using the Drucker-Prager material model. The raft and piles were modelled as linearly elastic materials. The soil and piles were modelled using solid continuum 8-node brick elements with reduced integration (element type C3DR). The raft was modelled using 3D shell elements (element type S8R). The number of nodes, elements and degrees of freedom were of the order of 8700, 6650 and 28000, respectively.

The raft-soil interface was modelled using "hard" contact model with tension cut-off. The interface between raft and piles was modelled using "TIE" contact. The "TIE" contact allowed no relative displacement between the raft and piles at the interface. The interface between the pile shaft to soil was modelled as perfectly rough, no relative motion between the pile shaft and soil element in contact with pile shaft was allowed. In this case, no special elements were used to model the interface; instead the thickness of the soil elements at the interface was kept at 0.1 times pile width. Around the pile a smeared zone equal to half pile width on each

side was created and reduced strength and stiffness properties as shown in Table 9.1 were prescribed for this zone of soil. Both the piles and soil were modelled without consideration of installation effects. A central point load of 40 kN was applied on the raft in 160 increments each of 0.25 kN.

Figure 9.1 Finite element mesh pattern for piled raft specimens for 9-pile piled raft with piles of $B = 30$ mm & $L = 200$ mm

9.2.1 Settlement response of model piled raft 9PB3L2

The central, average and differential settlement response obtained from FE analyses is discussed in this section. The nodal displacement at the centre of the raft in the z -direction was taken to be the central settlement of the raft. The average settlement of the raft was calculated by taking the area weighted average of the displacement of all the nodes of the raft in z -direction. The true maximum differential settlement in this case occurred between the nodal displacements at the centre and at the corner of the raft. However, in the model tests, the 'corner' settlement was measured at a location approximately 30 mm along the diagonal away from the corner. Hence for direct comparison, the computed settlement at the corresponding location was used instead.

Figure 9.2 Applied central point load (P) versus central settlement (δ_c) response of 9PB3L2 obtained from FE analyses and model test

Central settlement

The applied load (P) versus central settlement (δ_c) response of the model piled raft 9PB3L2 predicted numerically is plotted in Figure 9.2(a). There was general agreement in the computed and experimental curves. For $P < 18$ kN, the FE analysis yielded higher settlement. The trend was reversed for $P > 18$ kN.

However, there was significant difference between the experimental and computed response for the unpiled raft. The computed settlement for the unpiled raft was only

slightly larger than the piled raft's. The major discrepancy between the computed and experimental reduction in settlement (Figure 9.2(b)) could be traced to the disagreement in the response obtained for the unpiled rafts.

Figure 9.3 Applied central point load (P) versus average settlement (δ_{avg}) response of 9PB3L2 obtained from FE analyses and model test

Average settlement

In Figure 9.3, the P - δ_{avg} response obtained from FE analysis is compared with measured data. Figure 9.3 shows that the computed and measured responses tracked each other very closely for P up to 18 kN. Beyond this value, a much stiffer P - δ_{avg} response was obtained from FE calculations.

The computed bearing capacity (P_u^*) of the model piled raft 9PB3L2 interpreted using the two tangent method and the $0.1B_R$ method from the numerical P - δ_{avg} curve was 31.0 kN and 34.5 kN, respectively. From the experiment, the corresponding values for P_u^* were 25.0 kN and 30.0 kN, respectively. The predicted P_u^* from FE analysis was thus about 24% and 15% higher than the experimental values.

Differential settlement

Examination of the computed deformed shape of the raft showed that it was similar to that obtained in the experiment, with highest settlement at the centre and smallest settlement at the corner. The measured and computed differential settlements, $\Delta\delta$, were compared in Figure 9.4(a). The computed $\Delta\delta$ was higher than the measured value. The difference in the computed and measured differential settlement stayed relatively constant and did not appear to be significantly affected by changes in applied load.

The variation of reduction in $\Delta\delta$ with P , with the $\Delta\delta$ in unpiled raft serving as the base of reference, is shown in Figure 9.4(b). The computed percent reduction in $\Delta\delta$ was smaller than the experimental value. The highest difference was observed at low P . At $P = 5$ kN, the difference was about 30%, but reduced with P and vanished when P exceeded 15 kN.

In summary, in the 3-D finite element analyses the central and average settlement was predicted with reasonable accuracy up to $P = 18$ kN; this load level was equivalent to 60% of the ultimate capacity of the unpiled raft computed using linear elasto-plastic material model for soil. For $P > 18$ kN, the FE computation yielded a much stiffer P - δ response. FE analysis over-predicted the bearing capacity of the model piled raft by about 15% to 24%, depending on the interpretation methods used. The lower figure corresponded to interpretation based the $0.1B_R$ method, and the higher figure was obtained by interpretation based on the two tangent method. The predicted and measured settlement troughs were largely similar. The differential settlement was slightly over-estimated in the FE analyses.

Figure 9.4 Applied central point load (P) versus differential settlement ($\Delta\delta$) response of 9PB3L2 obtained from FE analyses and model test

9.2.2 Distribution of applied load between raft and piles

Mobilisation of pile head loads

The loads in the piles were calculated based on the average of the axial stresses at integration points of all the elements representing the pile head. The mobilisation of pile head loads with P of individual piles obtained numerically was compared with the measured data in Figure 9.5. For P up to 10 kN, the pile head loads were mobilised at a faster pace in the FE analysis compared to the measured data. For P

> 10 kN, the computed and measured data showed reasonable agreement. The numerical results also showed that the central pile attracted a much higher load than the edge and corner piles. The differences in the edge and corner piles were small. This trend also existed in the experimental data, but less well-differentiated than the FE output. As full mobilisation of the pile capacities was approached, all the piles converged to a pile head load approximately equal to the single pile capacity. The scatters in the curves were small. Figure 9.5 also indicated that full mobilisation of pile capacities occurred at about $P = 10$ kN. This value should be borne in mind when discussing the development of interface contact pressure which will be addressed in the next section.

Figure 9.5 Variation of mobilised geotechnical resistance of individual piles of $B = 30$ mm and $L = 200$ mm 9-pile piled raft obtained from FE analyses and model test

9.2.3 Load transfer through raft bearing pressure

As mentioned earlier, in the 3-D finite element analyses the raft-soil interface was modelled using "hard" contact model with tension cut-off. By using the "hard" contact model Abaqus automatically generated the raft-soil interface contact pressure. The raft-soil interface contact pressure thus obtained had been validated against standard bench mark solutions. The results have been described in Chapter 8. It was observed that "hard" contact model yielded reasonably accurate interface contact pressures. In this section the interface contact pressure and bearing resistance obtained from FE analyses are compared with the experimental results.

Interface contact pressure at quarter point

The interface contact pressure (σ_c) from FE analyses at quarter points is compared with the measured value in Figure 9.6. The results showed that the numerical data exhibited a significant 'lag' in the contact pressure response, with a very slow build-up initially. In contrast, the experimental data showed an approximately linear increase of contact pressure with P right from the beginning. In the FE calculations, the initial near zero gradient in the p_a - σ_c curve up to $p_a = 50$ kPa indicated that there was very little load transfer through the raft-soil contact at quarter points. The contact pressure only began to pick up with accelerated rate for $p_a > 50$ kPa.

For $p_a > 110$ kPa (or $P > 10$ kN; i.e. after piles had nearly mobilised their full capacities) the rate of increase in σ_c with applied load, represented by the gradient of the slope, was similar to that of the unpiled raft. Similar trend was also observed in the model test, though the latter showed some minor fluctuations.

Overall, the FE computations showed that the inclusion of piles had led to a greater reduction in the interface contact pressure. The computed and measured response for the unpiled raft was comparable.

Figure 9.6 Mobilisation of interface contact pressure (σ_c) with applied load (P) at quarter points 9-pile piled raft with $B = 30$ mm and $L = 200$ mm obtained from FE analyses and model test

Interface contact pressure at corner point

In the model tests, the total stress transducers were installed at a location along the diagonal situated at 30 mm from the raft corner. Therefore, the contact pressure from the finite element analyses at the corresponding location was used for comparison with the measured results. Figure 9.7 shows the computed and experimental p_a - σ_c curves at the raft corner. From the FE results, the piles did not seem to cause major changes in σ_c in contrast to the unpiled raft for p_a up to 100 kPa. When p_a exceeded 100 kPa, the curve for the unpiled raft showed an unusual response, with a short, nearly flat, segment for p_a between 120-180 kPa when σ_c hardly changed with increase in P . This rather untypical response could be due to issues related to FE discretisation and the high stress concentrations that existed near the sharp raft corners. Such anomalous response was not observed in the response in the interior quarter points.

If the curve for the unpiled raft was to be adjusted for the kink, the trend appeared to suggest that the contact pressure at the corner of the piled raft was not significantly changed by the presence of piles. This was in contrast to the experimental data which showed significant reduction in contact pressure in the piled raft.

Based on the $\sigma_c - p_a$ response presented in Figures 9.6 and 9.7, it could be deduced that the contact pressure distribution predicted by FE analysis varied more rapidly across the raft interface than was measured in the experiments. FE analyses yielded a much higher pressure gradient compared to the model tests.

Figure 9.7 Mobilisation of interface contact pressure (σ_c) with applied load (P) at corner points 9-pile piled raft with $B = 30$ mm and $L = 200$ mm obtained from FE analyses and model test

Raft bearing resistance

The variation of raft bearing resistance with applied load P is presented in Figure 9.8. The bearing resistance was obtained by multiplying the weighted area average

of contact pressure at all nodes with net raft-soil contact area computed as the gross raft area minus the area occupied by piles. The computed initial gradient of the raft bearing resistance versus P curve was much smaller than the measured value. However, as P increases, the computed gradient increases more rapidly than the measured data. Notwithstanding the small fluctuations, the computed gradient appears to stabilise at large P , a trend which was also observed in the test data. The FE analysis showed a smaller contribution from raft bearing resistance than the experimental data at low P , but the role was reversed at large P .

Figure 9.8 Variation of raft bearing resistance with applied load in 9-pile piled raft with $B = 30$ mm and $L = 200$ mm obtained from FE analyses and model test

9.2.4 Load sharing between raft and piles

The load distribution between piles and raft is shown in Figure 9.9. Ideally, the pile loads and the raft bearing resistance should add up to 100% of the applied load. This was approximately true for FE calculations when $P < 10$ kN. Beyond that, the FE results showed an error of up to + 7%, while the test results showed a larger discrepancy of -15%.

The FE analysis yielded a much higher contribution from piles of up to 80% in the beginning, dropping to around 30% at the end of computation. In comparison, in the model test the maximum pile contribution obtained varied from slightly less than 60% to around 28% towards the end of the test. For $P > 12$ kN, the computed and experimental curves tracked each other closely. The convergence of the two curves coincided with the initiation of full mobilisation of pile capacities.

The computed raft bearing load neatly mirrored the changes in pile loads, particularly for $P < 12$ kN. The percentage contribution from raft bearing pressure increased from a low of 20% initially to nearly 80% at the end of computation. In contrast, the model test results showed a smaller range of variation, increasing from 40% to 60%.

Figure 9.9 Variation of percentage load carried by piles and raft with the applied load (P) in 9-pile piled raft with piles of $B = 30$ mm and $L = 200$ mm obtained from FE analyses and model test

9.3 Factors Affecting Settlement of Piled Raft

As discussed in preceding chapters, the experimental load-settlement response of piled rafts was dependent on various pile parameters including quantities, length and width, and pile spacings. There were indications from the test results that piles of similar bearing capacity and stiffness but different geometry tended to yield similar piled raft response in typical working load range. The working load range is taken to mean an applied load that was well below the ultimate bearing capacity of the corresponding unpiled raft. The effects of these pile parameters were investigated in the FE analysis. The computed output would be compared with the experimental data.

9.3.1 Effect of number of piles

The computed $P - \delta_c$ response of 4PB3L2, 9PB3L2, 16PB3L2 and unpiled raft obtained is shown in Figure 9.10(a). The curves obtained for various piled rafts were similar in shape to the unpiled rafts, but with stiffness increasing with the number of piles – graphs shifting to the right in the figure with increasing pile number. At large P , the curve for 4PB3L2 converged with curve representing the unpiled raft.

The effect of the piles was better illustrated by the percentage reduction on the piled raft using the unpiled raft response as the base of reference, as shown in Figure 9.10(b). The percent reduction in raft settlement computed by FE analysis was lower than values obtained from the model tests.

Figure 9.10 Applied central point load (P) versus central settlement (δ_c) of 4-, 9- and 16-pile piled rafts with piles of $B = 30$ mm and $L = 200$ mm obtained from FE analyses

9.3.2 Effect of length of piles

Figure 9.11(a) shows the FE computed P - δ_c response of two pairs of 4- and 9-pile piled rafts with piles of different lengths. The load-settlement curves showed that increasing the pile length increased the piled-raft stiffness and ultimate bearing capacity.

Figure 9.11 Load (P) versus central settlement (δ_c) of 4- and 9-pile piled rafts with piles of $B=20$ mm, $L=200$ mm & 400 mm obtained from FE analyses

The percent reduction of the settlement is illustrated in Figure 9.11(b). The FE results showed that at low P , the effect of pile length was small, becoming more pronounced when $P > 18$ kN, a load level that corresponded to the onset of significant nonlinearity in the load-settlement curves [Figure 9.11(a)]. Both the computed and test results showed that pile length exerted a very significant influence on the raft settlement.

9.3.3 Effect of pile dimensions for piles having similar capacity and stiffness

The load tests performed on the model piles B3L2 and B2L4 yielded similar ultimate capacities and stiffness. This was also confirmed in the FE analyses of the singles piles. Figure 9.12 shows the computed P - δ_c response of 4-pile and 9-pile piled rafts equipped with B3L2 and B2L4 piles. The results show that at low applied load level of P up to about 10 kN, both the 4-pile and 9-pile piled rafts exhibited very similar response, as indicated by the plot of settlement reduction in Figure 9.12(b). The impact of pile geometry became evident only when $P > 18$ kN, when the longer piles began to induce a greater reduction in raft settlement. Raft 9PB2L4 showed the stiffest response and the largest ultimate bearing capacity. A comparison of the computed and experimental results again showed that FE analyses under-estimated the percent settlement reduction.

Figure 9.12 Load (P) versus central settlement (δ_c) of 4- and 9-pile piled rafts with piles of the same capacity but different geometry

9.4 Distribution of Applied Load between Raft and Piles

Mobilisation of pile resistance

Figure 9.13 shows the computed mobilisation of pile head loads with applied load (P) in 4PB3L2, 9PB3L2, 16PB3L2 rafts obtained from FE analyses. The piles used in these rafts were identical, 30 mm wide and 200 mm long. The tests on model piled rafts had shown that increasing the number of piles in the raft tended to delay the mobilisation of pile resistance, similar response was also obtained in the FE analysis with one exception. The exception being the centre pile (pile 1) in the 9PB3L2 which showed a faster rate of resistance mobilisation than 4PB3L2.

Figure 9.13 Variation of mobilised geotechnical resistance of individual piles of $B = 30$ mm and $L = 200$ mm in 4-, 9- and 16-pile piled rafts obtained from FE analyses

Variation of raft bearing resistance with applied load

The computed variation of raft bearing resistance with applied load is shown Figure 9.14(a). The model test results are shown in Figure 9.14(b). There was a broad similarity in the overall response curves obtained from FE analysis and the model tests. On closer examination, it should be obvious that there was significant difference. One prominent difference was the marked delay in the mobilisation of the bearing resistance of the raft 16PB3L2 obtained in FE analysis.

Load distribution between raft and piles

Figure 9.15 shows the computed variation of percentage load distribution between the piles and raft with applied load. As to be expected, the percentage load borne by piles increased with increasing pile number. This was particularly prominent in 16PB3L2 in which nearly 100% of the applied load was carried by piles, the raft bearing resistance became apparent only for $P > 5$ kN. Raft 9PB3L2 initially carried about 80% of the applied load, dropping to around 30% at $P = 25$ kN. In the 4PB3L2, the corresponding figures are 40% and 18%, respectively. The gradual drop in pile load contribution was accompanied by a gradual pick-up in the independently calculated raft bearing resistance. The sum of the pile load and bearing resistance was also presented in Figure 9.15, which deviated from the theoretical 100% by a small amount.

Figure 9.14 Variation of raft bearing resistance with applied load in 4-, 9- and 16-pile piled rafts with $B = 30$ mm and $L = 200$ mm obtained from FE analyses and model tests

Figure 9.15 Variation of percentage load carried by piles and raft with the applied load (P) in 4-, 9- and 16-pile piled raft with piles of $B = 30$ mm and $L = 200$ mm obtained from FE analyses

9.5 Contact Pressure versus Settlement Response

In this section, the effect of pile parameters on computed interface contact pressure would be examined. The contact pressure versus displacement relationship represents the so called 'soil-spring' response at the raft-soil interface. The information derived from the computation could be used to assess the applicability of representing the response of the soil continuum by the much simpler discrete soil springs in simplified numerical approaches.

Mobilisation of interface contact pressure at quarter points

The interface contact pressure (σ_c) versus displacement (δ) curves at the raft-soil interface at quarter points in four 4-pile piled raft and the unpiled raft obtained from FE analysis are presented in Figure 9.16. The model tests results were reproduced in Figure 9.16(b) for ease of comparison.

(a) numerical

(b) experimental

Figure 9.16 Interface contact pressure (σ_c) versus settlement (δ) response at quarter points for all 4-pile piled rafts obtained from FE analyses and model tests

Figure 9.16(a) shows that the $\sigma_c - \delta$ curves for raft with the piles of the same width (but different length) could not be clearly differentiated. Wider pile width appeared to contribute to a slightly softer response, the difference become more apparent when σ_c exceeds 100 kPa. The curves associated with narrower pile tracked the unpiled raft response closely throughout the loading range.

There was significant qualitative difference in the computed and experimental curves. In particular, the experimental data exhibited marked non-linearity right from the beginning; whereas the computed curves showed a significant initial linear response. The concavity of the experimental curves as contrasted to the sustained linear response in the computed results give rise to significant differences in magnitude of the interface contact pressure. The FE results showed maximum σ_c exceeding 240 kPa, while the test results yielded σ_c that did not exceed 160 kPa.

The above observation was confirmed in the FE analysis of 9-pile piled rafts plotted in Figure 9.17. The pairs of rafts with the same pile width showed essentially the same response when $\delta < 3$ mm. Again the pile width effect became apparent only with the onset of nonlinearity when $\delta > 5$ mm. Wider pile also caused a noticeable delay in the development of interface contact pressure regardless of the pile length.

However, the effect of pile width was not apparent in the model test results, with the curves for all four model rafts tightly clustered together when σ_c was less than 80 kPa. Another major point of departure between the computed and experimental results was the magnitude of σ_c relative to the value obtained in the unpiled raft. The experimental $\sigma_c - \delta$ curve was found to lie above the curve for the unpiled raft. In contrast, the FE computed curves for piled rafts were found to lie below the curve for the unpiled raft. However, unlike the 4-pile piled rafts, the difference in the maximum computed and measured σ_c was less stark.

(a) numerical

(b) experimental

Figure 9.17 Interface contact pressure (σ_c) versus settlement (δ) response at quarter points for all 9-pile piled rafts obtained from FE analyses and model tests

Figure 9.18 provides an even more vivid illustration of the dissimilarity between the computed and experimental σ_c - δ curves. The computed curves for 4PB3L2, 9PB3L2, and 16PB3L2 were spaced far apart, denoting very strong influence of pile number on the development of raft contact pressure. The computed piled raft responses were softer than the unpiled raft's. The response of the 4PB3L2 tracked the unpiled raft response closely for the entire range of applied load.

Three major observations could be made concerning the model test results that highlighted the differences in the computed and measured response. Firstly, the curves for all 4 rafts were practically indistinguishable for $\delta < 2$ mm. Secondly, when δ was greater than 2 mm, the result of the 4PB3L2 was practically indistinguishable from the unpiled raft response. The curves for 9PB3L2 and 16PB3L2 overlapped each other for nearly the entire range of applied load, whilst the curve for the 4PB3L2 overlapped the unpiled raft. The difference in the experimentally determined contact pressure for all rafts at any given displacement paled when compared to computed results obtained from the FE analysis. Finally, all the experimental curves showed distinct concavity downwards, with the gradient of the graph diminishing monotonically with increasing P , the reverse trend was observed in all the computed curves.

Figure 9.18 Interface contact pressure (σ_c) versus settlement (δ) response at quarter points for all 4-, 9- and 16-pile piled rafts with piles of $B=30\text{mm}$ & $L=200\text{ mm}$ obtained from FE analyses and model tests

Mobilisation of interface contact pressure at corner points

The computed contact pressure (σ_c) versus displacement (δ) curves at the raft corners of all the 4-pile and 9-pile piled rafts obtained from the FE analysis are consolidated in Figures 9.19 and 9.20. The model test results have been reproduced in the same figures for comparison.

The computed σ_c - δ curves for piled rafts were slightly stiffer than that of the unpiled raft. As mentioned previously with regards to the unpiled raft, the σ_c - δ curves also showed a curious kink at $\sigma_c = 100$ kPa, although it was less prominent than the one observed for unpiled raft. As a result, the curves for the piled raft deviated quite significantly from the unpiled raft for $\sigma_c > 100$ kPa. This kink could be an artefact of the 3-D FE analysis associated with mesh discretisation and high stress gradients at raft corners.

There were some evidence of the effect of pile length in the computed results, but the difference was small and was only observable for σ_c between 100 kPa to 200 kPa. The effect of the pile width was more clearly visible in the computed σ_c - δ curves for the four 9-pile piled rafts (Figure 9.20(a)), with wider piles producing a stiffer response. No kink was observed in the computed curves for the 9-pile piled rafts. The computed σ_c - δ curves for all the unpiled rafts converged to the same ultimate pressure of about 240 kPa or approximately $6c_u$.

No kink was observed in the model test results. The experimental curves were also closely clustered at low σ_c level. The rafts with short piles tracked the unpiled raft response closely, even at high σ_c level. The two rafts with long piles show a ‘softer’ response at high σ_c level.

Once again, a comparison of the computed and test results presented in Figures 9.19 and 9.20 showed that the FE computed ‘soil spring’ relationship at the raft-soil interface was significantly different from those obtained in the model piled rafts tests.

Figure 9.19 Interface contact pressure (σ_c) versus settlement (δ) response at quarter points for all 4-pile piled rafts obtained from FE analyses and model tests

(a) numerical

(b) experimental

Figure 9.20 Interface contact pressure (σ_c) versus settlement (δ) response at quarter points for all 9-pile piled rafts obtained from FE analyses and model tests

Comparison of σ_c - δ curves at quarter points and corner of the raft

The computed σ_c - δ curves at the corner and quarter point of the piled rafts are plotted in Figure 9.21. Only the data for the unpiled raft, 9PB3L2 and 16PB3L2 are shown.

The computed σ_c - δ curves at the corner and at quarter point in the unpiled raft showed only minor differences. The inclusion of piles caused a major change in the σ_c - δ response at the two raft locations. The response at corner was much stiffer than that at the quarter point. Pile number did not seem to affect the σ_c - δ curve at the corner at low applied load level. However, it drastically modified the response curve at the quarter point, especially in terms of the stiffness in the initial stages.

The model test results confirmed that the σ_c - δ curves at the two raft locations were dissimilar, but the effect was much less significant than that suggested by FE calculations. In contrast, for the unpiled raft, the test data showed a larger difference in the σ_c - δ curves at the two locations than those obtained from FE analysis.

In summary, significant differences in the computed and measured σ_c - δ , both qualitatively and quantitatively. FE results suggested that the number of piles and pile width would exert strong influence on the σ_c - δ curves; and a smaller impact of pile length effect, which became apparent only at large P . In contrast, the model tests indicated that pile parameters such as pile width, pile length and number of piles had only small to moderate influence on the σ_c - δ response; particularly when σ_c was less than 60 kPa.

(a) numerical

(a) experimental

Figure 9.21 Interface contact pressure (σ_c) with settlement (δ) at quarter points and corners in the unpiled raft, 9PB3L2 and 16PB3L2 obtained from FE analyses and model tests

9.6 Load-Settlement Response of Individual Piles in Piled Rafts

In this section the load-settlement behaviour of piles in the piled raft obtained from FE analyses would be examined and verified against the model test data.

Figure 9.22 shows the pile head load versus displacement responses of pile B2L2 acting in isolation and when it was incorporated in piles rafts. This pile had the narrowest width and shortest length among the 4 pile types used in the model tests. FE analysis indicated that the initial stiffness of the pile would be reduced in the piled raft [Figure 9.22(a)]. The computed responses of the piles in the raft were practically identical, except for the corner pile in the 9-pile piled raft. The ultimate capacities of the piles in the raft are smaller than single pile capacity.

The computed response shown in Figure 9.22(a) was significantly different from the measured response obtained in the model tests plotted in Figure 9.22(b). The model tests showed that the load-displacement curves of the piles were significantly affected both by the pile number and the relative position of the pile within the raft.

A similar set of data is presented in Figure 9.23 for piles B3L2 which had a larger width. The FE computed responses were qualitatively similar to those obtained for the B2L2 piles. There was better differentiation in the effect of relative pile position. There also appeared to be better agreement with the experimental results shown in Figure 9.23(b). Both the computed and experimental curve showed that the corner pile in the 9-pile piled raft attracted the highest load.

The response of pile B3L2 in the 16-pile piled raft is shown in Figure 9.24. For this piled raft, there was a better match between the computed and experimental curves. The corner pile was found to attract the largest load, followed by the edge pile (pile 2) in both the numerical and experimental data.

(a) numerical

(b) experimental

Figure 9.22 Pile head load versus pile head displacement response of pile B2L2 ($B = 20$ mm & $L = 200$ mm) single isolated case, in 4- and 9-pile piled rafts obtained from FE analyses and model tests

(a) numerical

(b) experimental

Figure 9.23 Pile head load versus pile head displacement response of pile B3L2 ($B = 30$ mm & $L = 200$ mm) single isolated case, in 4- and 9-pile piled rafts obtained from FE analyses and model tests

(a) numerical

(b) experimental

Figure 9.24 Pile head load versus pile head displacement response of pile B3L2 ($B = 30$ mm & $L = 200$ mm) single isolated case, in 16-pile piled rafts obtained from FE analyses and model tests

(a) numerical

(b) experimental

Figure 9.25 Pile head load versus pile head displacement response of pile B2L4 ($B = 20$ mm & $L = 400$ mm) single isolated case, in 16-pile piled rafts obtained from FE analyses and model tests

(a) numerical

(b) experimental

Figure 9.26 Pile head load versus pile head displacement response of pile B3L4 ($B = 30$ mm & $L = 400$ mm) single isolated case, in 16-pile piled rafts obtained from FE analyses and model tests

Figures 9.25 and 9.26 show the load-displacement curves for piles B2L4 and B3L4. A major conclusion that could be drawn from both the FE analysis and the model test data was that arising from the complex pile-soil and raft-soil interaction, the pile stiffness would be reduced in a piled raft. This reduction was likely to increase with increasing pile number. The relative location of the pile would affect the stiffness response. In fact, from the limited model raft tests conducted in this study, the effect arising from pile location was comparable to the effect due to pile number. However, it should be mentioned that the large divergence in the individual pile load-displacement response was observed only in rafts with narrow, short B2L2 piles. The divergence due to relative pile position was not observed in the FE analysis results.

9.7 Settlement Ratio (R_s)

As discussed in Chapter 7 and the preceding paragraphs, the load-settlement response of a pile would be significantly modified when it was included in the raft. The experimental and numerical results indicated that this response was dependent on the number of piles and pile spacing, among many other factors.

Poulos & Davis (1980) have proposed a method to relate the settlement of single pile and the average settlement of pile group through a factor termed the settlement ratio (R_s). R_s was defined as the ratio of the average group settlement $\bar{\delta}_g$ to the settlement of single individual pile at the same average load as a pile in the free standing pile group (δ_s).

$$R_s = \frac{\bar{\delta}_g}{\delta_s} \quad \text{Eqn (9.1)}$$

Where: $\bar{\delta}_g$ = average group settlement,

δ_s = settlement of single individual pile at the same average load as a pile in the free standing pile group

In the present study, to relate the settlement of pile in the piled raft with the settlement of single individual pile, the settlement ratio (R_s) was redefined as the

ratio of the settlement of a pile in the piled raft at a given load to the settlement of the single pile at the same load. That is, $\bar{\delta}_g$ in Eqn (9.1) was the settlement of pile in the piled raft at a given load and δ_s was the settlement of single individual pile at the same load. The ultimate capacity for the single isolated piles (Q_u) obtained from FE analyses are summarised in Table 8.5. The pile head load versus settlement curves for standalone single piles and piles in rafts presented in Figures 9.22 to 9.26 was approximately linear for a load corresponding to a factor of safety (FS) of 2 on the ultimate load for the single pile. Hence, in the present study the load corresponding to a FS of 2 on the ultimate load for single pile was used in the determination of R_s . The value of $\bar{\delta}_g$ in Eqn (9.1) was the settlement of a pile in the raft when the pile head load is equal to $Q_u/2$ and, similarly, δ_s was the corresponding settlement of the single pile at $Q_u/2$.

The variation of R_s with normalised pile spacing (s/d ratio) determined from the model tests data and FE computations are summarised in Table 9.2 and plotted in Figure 9.27. The plot shows that R_s from the model tests and FE data were in reasonable agreement. R_s obtained from FE analysis hovered around 2 for most cases. The largest R_s of 5 was obtained from one of the model tests.

Figure 9.27 Variation of settlement ratio (R_s) with s/d ratio

The R_s values derived by Poulos & Davis (1980) based on elastic theory for free standing pile groups with rigid cap had also been plotted in the figure. At a high s/d ratio of 6.5, the R_s values based on elastic theory for free standing groups showed reasonable agreement with both the model tests and FE results. However, at small s/d ratio, R_s proposed by Poulos & Davis was much higher than the values obtained in the tests and FE analysis.

9.8 Pile Load Contribution Factor (η)

As described in Section 7.4, the contribution from the piles may be quantified by a pile contribution factor, η . The η values for raft settlement $\delta = 1.0$ mm, 2.0 mm and 3.0 mm were calculated from the FE analysis results and summarised in Tables 9.3 to 9.5. The η values evaluated at $\delta = 3.0$ mm is shown in Figure 9.28. At $\delta = 3.0$ mm, the factor of safety (FS) against bearing capacity failure for the corresponding unpiled raft was about 3.0 based on P_u derived from the model tests data. FS based P_u derived from the numerical results was about 2.5.

Table 9.2 Settlement ratio (R_s) values from experimental and numerical study

S. No	Model Foundation	L (mm)	B (mm)	equivalent d (mm)	L//d	s/d	Exp		Poulos & Davis (1980)	Num	
							FS=1.25	FS=2.0		FS=1.25	FS=2.0
1	4PB2L2	200	20	22.6	8.85	6.65	2.28	1.59	1.76	3.56	1.79
2	4PB3L2	200	30	33.9	5.9	4.43	1.38	1.52	2.02	2.21	1.50
3	4PB2L4	400	20	22.6	17.7	6.65	1.38	1.11	1.89	1.67	1.15
4	4PB3L4	400	30	33.9	11.8	4.43	2.39	1.32	2.05	4.19	1.31
5	centre	200	20	22.6	8.85	4.43	5.53	4.82	3.25	3.54	1.88
	mid-edge	200	20	22.6	8.85	4.43	3.75	3.36	3.25	3.42	1.96
	corner	200	20	22.6	8.85	4.43	3.72	3.27	3.25	2.10	1.67
6	centre	200	30	33.9	5.9	2.95	2.24	2.00	3.94	2.02	1.75
	mid-edge	200	30	33.9	5.9	2.95	2.00	1.70	3.94	1.78	1.64
	corner	200	30	33.9	5.9	2.95	1.54	1.31	3.94	1.63	1.50
7	centre	400	20	22.6	17.7	4.43	1.76	1.37	3.62	2.29	1.59
	mid-edge	400	20	22.6	17.7	4.43	2.02	1.48	3.62	2.80	2.00
	corner	400	20	22.6	17.7	4.43	1.43	1.19	3.62	1.53	1.33
8	centre	400	30	33.9	11.8	2.95	2.16	1.39	4.00	3.72	1.78
	mid-edge	400	30	33.9	11.8	2.95	2.89	1.39	4.00	4.47	1.92
	corner	400	30	33.9	11.8	2.95	1.96	1.32	4.00	2.02	1.53
9	centre	200	30	33.9	5.9	2.22	2.92	2.10	6.27	2.65	2.21
	mid-edge	200	30	33.9	5.9	2.22	2.92	1.83	6.27	2.31	2.11
	corner	200	30	33.9	5.9	2.22	2.11	1.52	6.27	1.78	1.68

Table 9.3 Contribution factor (η) at $\delta = 1.0$ mm; ($P_o = 4.10$ kN, available factor of safety against bearing capacity failure (FS) for unpiled raft $\cong 7.4$)

S/ N	Model foundation	No of piles (n)	$\frac{L}{B}$	$\frac{\text{spacing}}{B}$	Q_u (kN)	P_d (kN)	$P_d - P_o$ (kN)	$\eta = \frac{(P_d - P_o)}{n.Q_u}$
1	16PB3L2	16	6.7	2.5	0.7	7.54	3.44	0.31
2	9PB3L4	9	13.3	3.3	1.2	8.19	4.09	0.38
3	9PB3L2	9	6.7	3.3	0.7	6.33	2.2	0.35
4	9PB2L4	9	20	5	0.75	6.8	2.7	0.4
5	9PB2L2	9	10	5	0.45	5.52	1.42	0.35
6	4PB3L4	4	13.3	5	1.2	6.24	2.14	0.45
6	4PB3L2	4	6.7	5	0.7	5.31	1.21	0.43
7	4PB2L4	4	20	7.5	0.75	5.47	1.37	0.46
8	4PB2L2	4	10	7.5	0.45	4.93	0.83	0.46

Table 9.4 Contribution factor (η) at $\delta = 2.0$ mm; ($P_o = 8.21$ kN, available factor of safety against bearing capacity failure (FS) for unpiled raft $\cong 3.7$)

S/ N	Model foundation	No of piles (n)	$\frac{L}{B}$	$\frac{\text{spacing}}{B}$	Q_u (kN)	P_d (kN)	$P_d - P_o$ (kN)	$\eta = \frac{(P_d - P_o)}{n.Q_u}$
1	16PB3L2	16	6.7	2.5	0.7	12.54	4.33	0.39
2	9PB3L4	9	13.3	3.3	1.2	13.62	5.41	0.50
3	9PB3L2	9	6.7	3.3	0.7	11.18	2.97	0.47
4	9PB2L4	9	20	5	0.75	11.53	3.32	0.49
5	9PB2L2	9	10	5	0.45	10.15	1.89	0.47
6	4PB3L4	4	13.3	5	1.2	10.93	2.72	0.57
6	4PB3L2	4	6.7	5	0.7	9.94	1.73	0.62
7	4PB2L4	4	20	7.5	0.75	9.9	1.69	0.56
8	4PB2L2	4	10	7.5	0.45	9.46	1.25	0.70

Table 9.5 Contribution factor (η) at $\delta = 3.0$ mm; ($P_o = 12.31$ kN, available factor of safety against bearing capacity failure (FS) for unpiled raft $\cong 2.5$)

S/ N	Model foundation	No of piles (n)	$\frac{L}{B}$	$\frac{\text{spacing}}{B}$	Q_u (kN)	P_d (kN)	$P_d - P_o$ (kN)	$\eta = \frac{(P_d - P_o)}{n \cdot Q_u}$
1	16PB3L2	16	6.7	2.5	0.7	15.9	3.59	0.32
2	9PB3L4	9	13.3	3.3	1.2	18.49	6.18	0.57
3	9PB3L2	9	6.7	3.3	0.7	16.09	3.78	0.6
4	9PB2L4	9	20	5	0.75	16.93	4.62	0.68
5	9PB2L2	9	10	5	0.45	14.97	2.66	0.66
6	4PB3L4	4	13.3	5	1.2	15.87	3.56	0.74
6	4PB3L2	4	6.7	5	0.7	14.71	2.4	0.86
7	4PB2L4	4	20	7.5	0.75	14.74	2.43	0.81
8	4PB2L2	4	10	7.5	0.45	13.86	1.55	0.86

Figure 9.28 shows that η values obtained for 16PB3L2 was the smallest. With the raft dimension fixed at 300 mm by 300 mm, the larger pile number meant that the pile spacing would be reduced. For 16PB3L2, s/d was 2.5. The small η was a reflection of the greater interaction effect of piles spaced very closely; and hence the smaller pile load mobilised at a given pile head displacement.

Raft 4PB2L2 also yielded a low η value of 0.5 even though the piles were spaced reasonably far apart with an s/d of 7.5. As has been discussed previously, for this particular raft with short, sparsely spaced piles, the dominant mechanism that controlled load transfer was the bearing capacity failure mechanism of the raft with the short piles located fully within the failure soil zones. This rendered the piles ineffective, particularly when the bearing capacity failure condition was encroached.

Excluding these two piled rafts, a conservative estimate of η could be derived from the straight line shown in Figure 9.28 which provided a lower bound to the limited data.

Figure 9.28 Variation of contribution factor (η) of piles with s/B ratio

9.9 Bearing Capacity of Piled Rafts (P_u^*)

The applied load (P) versus average settlement (δ_{avg}) response for the model piled rafts obtained from numerical analyses is presented in Appendix C in Figure C-1 to Figure C-8. The deduced values of P_u^* based on the two tangent method and the $0.1B_R$ method are summarised in Table 9.6. The back-calculated values of α_{UR} and α_G are also presented in the Table 9.6.

Table 9.6 Bearing capacities of piled rafts and back calculated α_G and α_{UR} values

1	2	3	4	5	6	7	8	9	10	11	12	13	14
	No. of piles (n)	Q_u (kN)	$n.Q_u$ (kN)	Two tangent method				α_{UR}	0.1 B_R method				
				P_u^* (kN)	Q_g^* (kN)	α_G	$(P_u^* - Q_g^*)$ (kN)		P_u^* (kN)	Q_g^* (kN)	α_G	$(P_u^* - Q_g^*)$ (kN)	α_{UR}
Unpiled raft				27.5					30.5				
4PB2L2	4	0.45	1.80	26.5	2.1	1.2	24.4	0.89	27.0	2.16	2.4	24.8	0.81
4PB3L2	4	0.70	2.80	27.0	3.2	1.2	23.8	0.86	27.5	3.4	3.9	24.1	0.79
4PB2L4	4	0.75	3.00	27.5	3.3	1.1	24.2	0.88	28.0	3.4	3.9	24.6	0.81
4PB3L4	4	1.30	5.20	28.5	5.1	1.0	23.4	0.85	29.0	5.2	6.1	23.8	0.78
9PB2L2	9	0.45	4.10	30.5	5.1	1.2	25.4	0.92	34.0	5.6	6.1	28.4	0.93
9PB3L2	9	0.70	6.30	31.0	8.3	1.3	22.8	0.83	34.5	8.24	10.0	26.3	0.86
9PB2L4	9	0.75	6.80	34.0	9.0	1.3	25.0	0.91	37.0	9.25	10.2	27.8	0.91
9PB3L4	9	1.30	11.70	33.0	13.7	1.2	19.3	0.70	38.0	13.9	19.8	24.1	0.79
16PB3L2	16	0.70	11.20	29.5	14.0	1.3	15.5	0.56	31.5	14.3	25.4	17.2	0.56

The P_u^* of 4PB2L2 and 4PB3L2 deduced from the two tangent method was about 5% and 2% smaller than the bearing capacity of the unpiled raft. The P_u^* presented in Table 9.6 indicated that the piles improved the bearing capacity of the unpiled raft by 2% to 24%. The back-calculated α_G values assuming group efficiency of unity were about 1.0 to 1.3. The back-calculated α_{UR} values for the nine piled rafts ranged from 0.56 to 0.92.

In the numerical analyses using simple elasto-plastic material model, P_u^* deduced from the two tangent method over-estimated the experimental value by about 10% to 33%. The back-calculated α_{UR} and α_G values were in reasonable agreement with the experimental values, except for 16PB3L2.

9.10 Summary

The nine piled raft tests were modelled in FE analysis using Abaqus 6.4. The soil parameters were obtained by calibrating the FE analysis output against the test results obtained for the unpiled raft and single pile described in Chapter 8.

It was found that 3-D FE modelling was capable of reproducing the load-settlement curves of the piled rafts with reasonable accuracy. The FE analysis tended to give slightly smaller settlement than the model test results when the applied load was high, thus giving rise to higher ultimate bearing capacities. FE analysis also yielded slightly higher differential settlement.

Both FE analyses and model tests showed that the pile stiffness in a piled raft would be reduced when compared to the isolated single pile. The amount of stiffness reduction tended to increase as the pile number in the raft increases.

The pile load contribution factor (η) determined from FE analysis generally agreed with experimental data. With the exception of the 4-pile piled raft with short piles and the 16-pile piled raft in which the pile spacing was small, η was found to be no less than 0.75 at reasonable working load level. Reasonable working load in this context was taken to be one with a factor of safety of 2 to 3 defined using the ultimate bearing capacity of the unpiled raft as the base of reference.

Although there were reasonable agreements between the computed and measured macro level response such as the load-displacement curves, there were major differences in other detailed behaviour. The FE simulations gave higher pile stiffness within the piled raft in the early stages, giving rise to the piles carrying a higher proportion of the applied load than was observed in model tests. Consequently, the interface contact pressure obtained from FE modelling was much smaller than the measured response. The FE analyses also showed higher gradient between the interface contact pressure at quarter points and corner when compared to the model test data.

The stress-displacement response at the raft soil interface computed by FE analysis suggested that it would be significantly affected by pile number and pile width. This was at variant with the measured data. The computed soil spring response was found to differ from the test result both qualitatively, in terms of the shape of the curve; and also quantitatively, in terms of the magnitude of the contact pressure.

In conclusion, although FE analysis showed some agreement with the model test results for macro-level responses such as load-settlement relationship, there were significant differences in several other key aspects. The latter includes load distribution between piles and raft and the development of contact stress with raft displacement and the influence of pile width. Such discrepancies between FE analysis and model tests would warrant further study.

Chapter 10

Conclusions and Recommendations for Future Work

The mechanics of how piles modify the load-settlements characteristics and the ultimate load bearing capacity of the piled raft was the focus of this study. A total of nine tests were conducted on model piled rafts by varying pile number, piles width and pile length in a relatively rigid raft of fixed dimension. A comprehensive and careful analysis of the experimental data was carried out to unravel the effect of piles in piled rafts behaviour.

The piled rafts were also analysed using 3-D finite element analysis program Abaqus. The parameters used in the finite element analysis were derived by calibration exercise using experimental data derived from load tests on single piles and unpiled rafts. Detailed comparative study was carried out to compare the numerical output with the model test data to verify the validity and accuracy of 3-D finite element analysis of piled raft response.

Based on the experimental test data and also the numerical output obtained through 3-D finite element analysis, the following conclusions may be drawn.

10.1 Conclusions

Model piled raft tests

1. The number of piles in the raft was the primary factor that governed the settlement reduction, particularly when the piles were spaced far apart to reduce the effect of pile-soil-pile interactions. If increasing the pile number caused the pile spacing to encroach on the optimum threshold, the addition of more piles could only yield marginal and diminishing improvement in the load-settlement response of the raft.

2. The model test data showed that piled raft settlements were strongly influenced by the stiffness and load bearing resistance of the piles. Piles of the same capacity and stiffness, but with different pile geometry, had the same effect on raft settlement, particularly at applied load level which was well below the ultimate bearing capacity of the raft.
3. Short piles were effective in reducing raft settlements when the applied load was not large enough to induce significant yielding in the soil. Long, stiff piles were more effective and efficient in reducing raft settlements than short piles of similar cross section.
4. Increasing pile width increased the reduction in raft settlements at low applied load level insufficiently to trigger foundation soil failure, particularly when the piles were placed sufficiently wide apart.
5. Short piles were not effective in increasing the ultimate load bearing capacity of piled raft. Sparsely spaced long stiff piles were more effective in increasing the bearing capacity of the raft.
6. The experimental data showed that the interface contact pressure-displacement relationship (or soil spring curve) at the raft-soil interface was highly non-linear even at small raft displacement. The soil spring curve at corner was found to be stiffer than soil spring curve at quarter points.
7. At low stress level, the soil spring was unique and was independent of pile dimension and number of piles present in the raft. At higher stress level, the soil-spring curves at quarter point and corner of the raft would diverge slightly. Hence, when modelling piled raft response in a low to moderate stress level it may be reasonable to assume a unique non-linear soil spring.
8. Due to pile-raft and pile-soil-pile interactions, the stiffness of the pile within the raft was smaller than the stiffness of the isolated single pile. The experimental data showed that the stiffness of pile was significantly affected by the spacing between piles. The changes in the stiffness were more pronounced for short piles.

9. The model test data indicated that the load bearing capacity of the piles in piled raft were not significantly different from the capacity of standalone single pile.
10. A contribution factor (η) has been defined to quantify the effectiveness of piles towards settlement reduction; it is a measure of proportion of ultimate capacity mobilised at a particular raft settlement. From the experimental data, it was found that with the exception of the 16-pile piled raft in which the piles were very closely spaced and the 4-pile piled raft with short slender pile, an η value of at least 0.75 had been achieved in all piled rafts.

Finite element analyses

1. The load-settlement response predicted by FE analysis showed general agreement with the measured test data with reasonable accuracy within the range of working loads. The FE analysis gave slightly smaller settlements at higher load level; it over-estimated the ultimate bearing capacity.
2. Though the FE analysis predicted load-settlement response reasonably well, the predicted load distribution between piles and raft and the predicted raft-soil interface response were very different from those obtained in the experiments.
3. The FE analysis showed that number of piles in the raft and the pile width would exert very strong influence on the development of raft soil contact pressure. The computed soil spring curves were very different from those derived from model tests, both qualitatively and quantitatively. The shape of the computed and experimental soil-spring curves showed very little resemblance; the computed contact stress was also much higher than the measured value.
4. Apart from the macro-level response such as load-displacement curve, the FE analysis conducted in this study was not able to reproduce the detailed raft-soil interface response and the load distribution between pile and raft.

There were myriad of possibilities as to why the discrepancies exist. Further study is needed to clarify the situation.

10.2 Recommendations for Further Study

1. In the present study, the effect of pile number and pile spacing could not be clearly segregated due to the constraints of time and resources available. Further study can be carried out by placing the same number of piles at different pile spacing. This could help to clarify the influence of each of these parameters. More experiments should be carried out by extending the range of parameters such as pile length, pile width and the dimension and stiffness of the raft to provide a more comprehensive data set. The experimental data described here was a fair contribution towards building up that database of knowledge.
2. Instrumentation of real piled rafts could be undertaken to evaluate pile resistance contribution factor η in full size piled raft. Such data would be more relevant for practising engineers who need concrete information on full size foundation to help in the design process and decision making.
3. This study was conducted for piled rafts in clay. The conclusions drawn might not be directly applicable to rafts founded in sand. Similar model tests in sand should be conducted to determine whether the conclusions made in this study are also applicable to piled raft in sand.
4. The piles in the model tests were installed through pre-formed holes. This was not truly representative of actual piled rafts. The effects of pile and raft installation on piled raft performance would deserved further study, both experimentally and numerically through more realistic modelling.
5. Though the FE analysis was able to reproduce the load-displacement curves with reasonable accuracy, there were major differences in the computed load-distribution between raft and piles as compared to the experimental results. The cause of the discrepancies was not entirely clear. Uncertainties remained concerning the performance and suitability of the interface model

at the pile-soil and raft-soil interface. It is not inconceivable that better agreement could be achieved by refinements in the finite element models. This would constitute a useful topic of further research.

References

- Anagnostopoulos, C., and Georgiadis, M. (1998). "A simple analysis of piled raft foundations." *Geotechnical Engineering Journal*, 29(1), 71-83.
- Borel, S. (2001). "The design of piled-raft foundations under lateral load." *15th International Conference on Soil Mechanics and Geotechnical Engineering*. Istanbul, Turkey, 847-850.
- Briaud, J. L. and Jeanjean, P. (1994). "Load Settlement Curve Method for Spread Footings on Sand and Horizontal Deformations of Foundations and Embankments." *ASCE Journal of the Geotechnical Engineering Division*, 2, 1774-1804.
- Brown, P. T. (1969). "Numerical analysis of uniform load circular raft on elastic layers of finite depth." *Géotechnique*, 19(2), 301-306.
- Brown, P. T., and Wiesner, T. J. (1975). "The behavior of uniformly loaded piled strip footings." *Soils and Foundations*, 15(4), 13-21.
- Brown, P. T., Poulos, H. G., and Wiesner, T. J. (1975). "Piled raft foundation design." *Proceedings of symposium on raft foundations*. Perth, Australia, 13-21.
- Burland, J. B., Broms, B. B., and de Mello, V. F. B. (1977). "Behavior of foundations and structures." *Proceedings of 9th International Conference on Soil Mechanics and Foundation Engineering*. Tokyo, 495-546.
- Burland, J. B., and Burbridge, M. C. (1984). "Settlement of foundations on sand and gravel." *Glasgow and West of Scotland Association Centenary Lecture, Thomas Telford, London*.
- Burland, J. B., and Karla, J. C. (1986). "Queen-Elizabeth-II-conference-center-Geotechnical Aspects." *Proceedings of the Institution of Civil Engineers Part 1-Design and Construction*, 80, 1479-1503.
- Burland, J. B. (1995). "Piles as settlement reducers." *18th Italian Cong. Soil Mech., Pavia*.

- Butterfield, R., and Banerjee, P. K. (1971a). "The problem of pile group-pile cap interaction. ." *Géotechnique*, 21(2), 135-142.
- Butterfield, R., & Banerjee, P. K. (1971b). "The elastic analysis of compressible piles and pile groups. ." *Géotechnique*, 21(1), 43-60.
- Cao, X. D. (1998). "Performance of raft foundations with settlement reducing piles." *MEng Thesis, Nanyang Technological University, Singapore*.
- Cerato, A. B. (2005). "Scale Effects of Shallow Foundation Bearing Capacity on Granular Material." *University of Massachusetts Amherst*.
- Chow, Y. K. (1986). "Analysis of vertically loaded pile groups." *Int. J. Numer. Anal. Methods Geomech.*, 10, 59-72.
- Chow, Y. K. (1987). "Iterative analysis of pile-soil-pile interaction." *Géotechnique*, 37(3), 321-333.
- Clancy, P., and Randolph, M. F. (1993). " An approximate analysis procedure for piled raft foundations." *Int. J. Numer. Anal. Methods Geomech.*, 17, 849-869.
- Clancy, P., and Randolph, M. F. (1996). "Simple design tools for piled raft foundations." *Géotechnique*, 46(2), 313-328.
- Coduto, D. P. (1999). *Foundation Design: Principles and Practices*: Prentice-Hall, Inc.
- Conte, G., Mandolini, A., and Randolph, M. F. (2003). "Centrifuge modeling to investigate the performance of piled rafts." *Proc., Geotech. Int. Seminar on Deep Foundations on Bored and Auger Piles, Van Impe and Haegeman, eds.*, 379-386.
- Cooke, R. W., Bryden-smith, D. W., Gooch, M. N., and Sillett, D. F. (1981). "Some observations of the foundation loading and settlement of a multi storey building on a piled raft foundation in London clay." *Proceedings of Instn. of Civil Engrs.*, 107(Pt 1), 433-460.
- Cooke, R. W. (1986). "Piled raft foundations on stiff clays-a contribution to design philosophy." *Géotechnique*, 36(2), 169-203.
- de Sanctis, L. (2001). "Modellazione ed analisi delle fondazioni su pali." *Universita di Napoli, Federico II*.

de Sanctis, L., Mandolini, A., Russo, G., and Viggiani, C. (2002). "Some remarks on the optimum design of piled rafts. ." *Geotechnical special publication, ASCE*, n116I, 405-425.

de Sanctis, L., and Mandolini, A. (2006). "Bearing capacity of piled rafts on soft clay soils." *Journal of Geotechnical and Geoenvironmental Engineering*, 132(12), 1600-1610.

de Sanctis, L. and Russo, G. (2008). "Analyses and performance of piled rafts designed using innovative criteria." *Journal of Geotechnical and Geoenvironmental Engineering*, 134(8), 1118-1128.

DeBeer, E. E. (1970). "Experimental Determination of the Shape Factors and The Bearing Capacity factors of Sand." *Géotechnique*, 20(4), 387-411.

Desai, C. S., Johnson, L. D., and Hargett, C. M. (1974). "Analysis of pile-supported gravity lock." *Geotechnical Engineering Journal, ASCE* 100(9), 1009-1029.

Fioravante, V. (1998). "Load transfer mechanism of piled raft foundation." *Centrifuge 98* (Kimura, Kusakabe and Takemura (editors), Balkema, Rotterdam), 495-500.

Fleming, W.G.K (1992). "A new method for single pile settlement prediction and analysis." *Géotechnique*, 42(3), 411-425.

Fleming, W. G. K., Weltman, A. J., Randolph, M. F., and Elson, W. K. (1992). *Piling engineering*, Glasgow, U.K.: Blackie Academic.

Franke, E., Lutz, B., and El-Mossallamy, Y. (1994). "Measurements and numerical modeling of high-rise building foundations on Frankfurt clay." *Geotechnical special publication, ASCE*, 40, 1325-1336.

Fraser, R. A., and Wardle, L. J. (1976). "Numerical analysis of rectangular rafts on layered foundations." *Géotechnique*, 26(4), 613-630.

Fraser, R.A. and Wardle, L.J. (1976). "Numerical analysis of rectangular rafts on layered foundations." *Géotechnique*, 26(4), 613-628.

Giroud, J. P. (1968). "Settlement of a linearly loaded rectangular area." *Journal of the Soil Mechanics and Foundations Division*, 94(SM4), 813-820.

- Gourvenec, S., Rudolph, M. and Kingsnorth, O. (2006). "Undrained Bearing Capacity of Square and Rectangular Footings." *International Journal of Geomechanics*, 6(3), 147-157.
- Griffiths, D. V., Clancy, P., and Randolph, M. F. (1991). "Piled raft foundation analysis by finite elements. " *Proc. 7th Int. Conf. on Computer Methods and Advances in Geomechanics, Cairns*, 1153-1157.
- Hain, S. J., and Lee, I. K. (1978). "The analysis of flexible raft-pile systems." *Géotechnique*, 28(1), 65-83.
- Hansbo, S. & Jendeby, L. (1983). "A case study of two alternative foundation principles: conventional friction piling and creep piling." *Väg-och Vattenbyggaren* 7-8, 29-31.
- Hansbo, S. & Källström, R. (1983). A case study of two alternative foundation principles. *Väg-och Vattenbyggaren* **7-8**, 23-27.
- Hansbo, S. (1984). "Techno-economic trend of subsoil improvement methods in foundation engineering." *Proc. the ECSMFE*, 3, 1333-1343.
- Hansbo, S. (1993). "Interaction problems related to installation of pile groups." *Proceedings of 2nd Int. Geotech. Sem. on Deep Foundations on Bored and Auger Piles, Ghent*, 59-66.
- Hansen, B. (1961). "A General Formula for Bearing Capacity." *Bulletin 11, Geoteknisk Institute Copenhagen*.
- Hartmann, F., and Jahn, P. (2001). "Boundary element analysis of raft foundations on piles." *Meccanica*, 36, 351-366.
- Hongladaromp, T., Chen, N. J., and Lee, S. L. (1973). "Load distributions in rectangular footings on piles. ." *Geotechnical Engineering Journal*, 4 (2), 77-90.
- Hooper, J. A. (1974). "Review of behavior of piled raft foundations."
- Horikoshi, K. (1995). "Optimum design of piled raft foundations." *PhD thesis, The University of Western Australia, Australia*.
- Horikoshi, K., and Randolph, M. F. (1996). "Centrifuge modelling of piled raft foundations on clay." *Géotechnique*, 46(4), 741-752.

- Horikoshi, K., and Randolph, M. F. (1998). "A Contribution to optimum design of piled rafts." *Géotechnique*, 48(3), 301-317.
- Horikoshi, K., and Randolph, M. F. (1999). "Estimation of overall settlement of piled rafts." *Soils and Foundations*, 39(2), 59-68.
- Horikoshi, K., Matsumoto, T., Hashizume, Y., Wantanabe, T., Fukuyama, H. (2003a). "Performance of piled raft foundations subjected to static vertical loading and horizontal loading." *International Journal of Physical Modelling in Geotechnics*, 3(2), 37-50.
- Horikoshi, K., Matsumoto, T., Hashizume, Y., Wantanabe, T. (2003b). "Performance of piled raft foundations subjected to dynamic loading." *International Journal of Physical Modelling in Geotechnics*, 3(2), 51-62.
- Houlsby, G. T., Cassidy, M. J. and Einav, I. (2005). "A generalised Winkler model for the behaviour of shallow foundations." *Géotechnique*, 55(6), 449-460.
- Katzenbach, R., and Reul, O. (1997). "Theme lecture: Design and performance of piled rafts." *14th International Conference on Soil Mechanics and Foundation Engineering. Hamburg, Germany*, 2253-2256.
- Katzenbach, R., Arslan, U., and Moorman, C. (1998). "Design and safety concept for piled raft foundations." *Deep Foundations on Bored and Auger Piles*, van Impe & Haegman (editors), Balkema, Rotterdam, 439-448.
- Katzenbach, R., Arslan, U., and Moorman, C. (2000). *Pile raft foundations projects in Germany: Design applications of raft foundations*, Hemsley J. A. Editor, Thomas Telford.
- Katzenbach, R., and Moormann, C. (2001). "Recommendations for the design and construction of piled rafts." *15th International Conference on Soil Mechanics and Geotechnical Engineering. Istanbul, Turkey*, 927-930.
- Katzenbach, R., and Moormann, C. (2003). "Instrumentation and monitoring of combined piled rafts (CPRF): state-of-the-art report." In: F. Myrvoll, ed., *6th International Symposium on Field Measurements in Geomechanics*. Oslo, Norway, 161-177.

Katzenbach, R., Schmitt, A., and Turek, J. (2005). "Assessing settlement of high-rise structures by 3D simulations." *Computer-Aided Civil and Infrastructure Engineering*, 20, 221-229.

Kim, K.N., Lee, S.-H., Kim, K.-S., Chung, C.-K., Kim, M.M., and Lee, H.S. (2001). "Optimal pile arrangement for minimizing differential settlements in piled raft foundations." *Computers and Geotechnics*, 28, 235-253.

Kitiyodom, P., and Matsumoto, T. (2002). "A simplified analysis method for piled raft and pile group foundations with batter piles." *Int. J. Numer. Anal. Methods Geomech.*, 26(13), 1349-1369.

Kitiyodom, P., and Matsumoto, T. (2003). "A simplified analysis method for piled raft foundations in non-homogeneous soils." *Int. J. Numer. Anal. Methods Geomech.*, 27(2), 85-109.

Kitiyodom, P., Matsumoto, T., and Kawaguchi, K. (2005). "A simplified analysis method for piled raft foundations subjected to ground movements induced by tunnelling." *Int. J. Numer. Anal. Methods Geomech.*, 29, 1485-1507.

Kitiyodom, P., Matsumoto, T., and Sonoda, R. (2008). "A post-analysis of a large piled raft foundation constructed using reverse construction method." In: W. F. Van Impe, ed., *5th International Symposium on Deep Foundations on Bored and Auger Piles (BAP-V)*. Ghent, Belgium, 127-133.

Kitiyodom, P., Matsumoto, T., Horikoshi, K., and Watanabe, T. (2005). "Analyses of vertical and horizontal load tests on piled raft models in dry models." *16th International Conference on Soil Mechanics and Geotechnical Engineering*. Osaka, Japan, 2005-2008.

Kuwabara, F. (1989). "An Elastic Analysis for Piled Raft Foundations in a Homogeneous Soil." *Soils and Foundations*, 29(1), 82-92.

Lee, I. K. (1993). "Analysis and performance of raft and raft-pile foundations in a homogeneous soil." *Proc. 3rd Int. Conf. Case Hist. in Geotech. Engng.*

Liang, F.Y., Chen, L.-Z., and Shi, X.-G. (2003). "Numerical analysis of composite piled raft with cushion subjected to vertical load." *Computers and Geotechnics*, 30, 443-453.

- Liu, J., Huang, Q., Li, H., and Hu, W. L. (1994). "Experimental research on bearing behaviour of pile groups in soft soil." *Proceedings of 13th ICSMFE*, 2, 535-538.
- Love, J. P. (2003). "The use of settlement reducing plies to support a flexible raft structure in West London." In: T. A. Newson, ed., *BGA International Conference on Foundations*. Dundee, Scotland, 531-540.
- Maharaj, D. K., and Gandhi, S. R. (2004). "Non-linear finite elements analysis of piled-raft foundations." *Proceedings of the institution of Civil Engineers-Geotechnical Engineering*, 157(3), 107-113.
- Mandolini, A., Russo, G., and Viggiani, C. (2005). "Pile foundations: Experimental investigations, analysis and design." *16th International Conference on Soil Mechanics and Geotechnical Engineering*. Osaka, Japan, 177-213.
- Matsumoto, T., Fukumura, K., Kitiyodom, P., Oki, A., Horikoshi, K. (2004a). "Experimental and analytical study on behaviour of model piled rafts in sand subjected to horizontal and moment loading." *International Journal of Physical Modelling in Geotechnics*, 4(3), 1-19.
- Matsumoto, T., Fukumura, K., Oki, A., Horikoshi, K. (2004b). "Shaking table tests on model piled rafts in sand considering influence of superstructures." *International Journal of Physical Modelling in Geotechnics*, 4(3), 20-37.
- Matsumoto, T., Nemoto, H., Mikami, H., Yaegashi, K., Arai, T., Kitiyodom, P. (2010). "Load tests of piled raft models with different pile head connection conditions and their analyses, *Soils and Foundations*, 50(1), 63-68.
- Mattes, N. S., and Poulos, H.G. (1969). "Settlement of single compressible pile." *JSMFE, ASCE* 95(SM1), 189-207.
- McManus, K. J., and Kulhawy, F. H. (1991). "A Cohesive soil for large-size laboratory deposits." *Geotechnical Testing Journal*, 14(1), 26-34.
- Meyerhof, G. G. (1951). "The Ultimate Bearing Capacity of Foundations." *Géotechnique*, 11(4), 301-332.
- Micahlowski, R. L. and Dawson, E. M. (2002). "Three-dimensional analysis of limit loads on Mohr-Coloumb soil." *Foundations of civil and environmental engineering*, 1, 37-147.

- Michalowski, R. L. (2001). "Upper-bound load estimates on square and rectangular footings." *Géotechnique*, 51(9), 787-798.
- Padfield, C. J., and Sharrock, M. J. (1983). "Settlement of structures on clay soils." *CIRIA: Special Publication 27*.
- Poulos, H. G., and Davis, E. H (1991). "Analysis of piled strip foundations." *Computer Methods and Advances in Geomechanics*, 183-191.
- Poulos, H. G. (1994a). "An approximate numerical analysis of pile raft interaction." *Int. J. Numer. Anal. Methods Geomech.*, 18, 73-92.
- Poulos, H. G. (1994b). "Alternative design strategies for piled raft foundations." *Proceedings 3rd Int. Conf. Deep Foundations, Singapore*, 239-244.
- Poulos, H. G. (2000). "Pile-raft interaction-alternative methods of analysis." *Developments in Theoretical Geomechanics, Smith & Carter (editors), Balkema, Rotterdam*, 445-463.
- Poulos, H. G. (2001). "Piled raft foundations: design and applications." *Géotechnique*, 51(2), 95-113.
- Poulos, H. G. and Davis, E. H (1980). *Pile foundation analysis and design*: John Willey.
- Prakoso, W. A., and Kulhawy, F. H. (2001). "Contribution to piled raft foundation design." *Journal of Geotechnical and Geoenvironmental Engineering*, 127(1), 17-24.
- Randolph, M. F. (1994). "Design methods for pile groups and piled rafts: state-of-the-art report." *Proc. 13th Int. Conf. Soil Mech. Found. Engng., New Delhi 5*, 61-82.
- Randolph, M. F. and Clancy, P. (1993). "Efficient design of piled rafts." *Deep Foundations on Bored and Auger piles, Balkema, Rotterdam*, 119-130.
- Randolph, M. F. and Wroth, C.P. (1979). "An analysis of the vertical deformation of pile groups." *Géotechnique*, 29(4), 423-439.
- Reul, O., and Randolph, M. F. (2002). "Study of the influence of finite element mesh refinement on the calculated bearing behavior of a piled raft." *Proc. 8th Int. Symp. Numer. Models Geomech., Rome*, 259-264.

- Reul, O., and Randolph, M. F. (2003). "Piled rafts in overconsolidated clay: comparison of in situ measurements and numerical analyses. ." *Géotechnique*, 53(3), 301-315.
- Reul, O., and Randolph, M. F. (2004). "Design strategies for piled rafts subjected to nonuniform vertical loading." *Journal of Geotechnical and Geoenvironmental Engineering*, 130(1).
- Reul, O. (2004). "Numerical study of the bearing behaviour of piled rafts." *International Journal of Geomechanics (ASCE)*, 4(2), 59-68.
- Russo, G., and Viggiani, C. (1998). "Factors controlling soil-structure interaction for piled rafts. ." *Dramstadt Geotechnics (Dramstadt University of Technology)*, (4), 297-322.
- Russo, G. (1998). "Numerical analysis of piled rafts." *Int. J. Numer. Anal. Methods Geomech.*, 22, 477-493.
- Russo, G., Viggiani, C., and de Sanctis, L. (2004). "Piles as settlement reducers: A case history " *Advances in geotechnical engineering, The Skempton conference, Thomas Telford, London*, 2, 1143-1154.
- Salgado, R., Lyamin, A. V., Sloan, S. W. and Yu, H. S. (2004). "Two- and three - dimensional bearing capacity of foundations in clay." *Géotechnique*, 54(5), 297-306.
- Schmertmann, J. H., Hartman, J. P., and Brown, P. R. (1978). "Improved strain influence factor diagrams." *ASCE Journal of the Geotechnical Engineering Division*, 104(8), 1131-1135.
- Selvadurai, A. P. S. (1979). "Displacement of a rigid circular foundation anchored to an isotropic elastic-half-space." *Géotechnique*, 29(2), 195-202.
- Shen, W. Y., and Teh, C. I. (2002). "Practical solution for group stiffness analysis of piles." *Journal of Geotechnical and Geoenvironmental Engineering*, 128(8), 692-698.
- Shield, R. T. and Drucker, D. C. (1953). "The application of limit analysis to punch indentation problems." *Journal of Applied Mechanics*, 20, 453-460.

- Sinha, J. (1997). "Piled raft foundations subjected to swelling and shrinking." *PhD thesis, University of Sydney, Australia*.
- Skempton, A. W. (1951). "The bearing capacity of clays." *Building Research Congress*. London, 180-189.
- Small, J. C., and Zhang, H. H. (2000). "Piled raft foundations subjected to general loadings." *Developments in Theoretical Geomechanics, Smith & Carter (editors), Balkema, Rotterdam*, 431-444.
- Small, J. C., and Liu, Hana L.S. (2008). "Time-settlement behaviour of piled raft foundations using infinite elements." *Computers and Geotechnics*, 35, 187-195.
- Smith, I. M., and Wang, A. (1998). "Analysis of piled rafts." *Int. J. Numer. Anal. Methods Geomech.*, 22, 777-790.
- Sonada, R., Matsumoto, T., Kitiyodom, P., Moritaka, H., and Ono, T. (2009). "Case study of apiled raft foundation constructed using reverse construction method and its post-analysis." *Canadian Geotechnical Journal*, 46, 1-18.
- Sommer, H., Tamaro, G., and De Benedittis, C. (1991). "Messeturm, foundations for the tallest building in Europe." *Proc. 4th DFI Conf, Stresa*, 139-145.
- Ta, L. D., and Small, J. C. (1996). "Analysis of piled raft systems in layered soils." *Int. J. Numer. Anal. Methods Geomech.*, 20, 57-72.
- Ta, L. D., and Small, J. C. (1997). "An approximation for analysis of raft and piled raft foundations." *Computers and Geotechnics* 20(2), 105-123.
- Ta, L. D., and Small, J. C. (1998). "Analysis and performance of piled raft foundations on layered soils-case studies. ." *Soils and Foundations*, 38(4), 145-150.
- Tamaro, G. J. (1996). "Foundation engineers: why do we need them?." *1996 Martin S. Kapp Lecture. New York: ASCE*.
- Terzaghi, K. (1943). *Theoretical soil mechanics*, New York: Wiley.
- Thaher, M., and Jessberger, H. L. (1991a). "The behaviour of pile-raft foundations, investigated in centrifuge model tests." In: H. Y. Ko, and F. G. McLean, ed., *1991 International Conf on Centrifuge*. Boulder, Co, 225-234.

- Thaher, M., and Jessberger, H. L. (1991b). "Investigation of the behaviour of pile-raft foundations by centrifuge modeling." *10th European Conf of Soil Mechanics and Foundation Engineering*. Florence, Italy, 597-603.
- Trautmann, C. H. and Kulhawy, F H. (1988). "Uplift load-displacement behaviour of spread foundations." *Journal of Geotechnical Engineering, ASCE*, 114(2), 168-183.
- Vesic, A. S. (1977). "Design of pile foundations." National Cooperative Highway Research Program, Transportation Research Board, Washington, D. C. Report 42.
- Viggiani, C., Russo, G., and Basile, F. (2000). "Non-linear analysis of pile groups - Discussion." *Proceedings of the institution of Civil Engineers- Geotechnical Engineering*, 143(4), 241-244.
- Viggiani, C. (2001). "Analysis and design of piled foundations. 1st Arrigo Croce Lecture," *Rivista Italiana de Geot.*, 1/2001, 47-75.
- Wang, A. (1996). "Three dimensional finite element analysis of pile groups and piled -rafts." *PhD thesis, University of Manchester, U.K.*
- Whitaker, T. (1957). "Experiments with model piles in groups." *Géotechnique*, 7(4), 147-167.
- Yamashita, K., and Kakurai, M. (1991). "Settlement behavior of the raft foundation with friction piles." *4th International DFI Conference, Balkema, Rotterdam*, 461-466.
- Yamashita, K., Kakurai, M., and Yamada, T. (1994). "Investigations of a piled raft foundation on stiff clay." *XIII ICSMFE, New Delhi*, , 543-546.
- Yamashita, K., Yamada, T. and Kakurai, M. (1998). "Method for analysing piled raft foundations." *Deep Foundations on Bored and Auger Piles, van Impe and Haggeman (editors), Balkema, Rotterdam*, 457-464.
- Yamashita, K., Yamada, T., and Kakurai, M. (1998). "Simplified method for analysing piled raft foundations." In: eds; Balkema Van Impe and Haegeman, Rotterdam, ed., *Deep Foundations on Bored and Auger Piles*.
- Yamashita, K., Yamada, T., and Hamada, J. (2009). "Recent case histories on monitoring settlement and load sharing of piled rafts in Japan." *Deep Foundations*

on Bored and Auger Piles, van Impe (editors), Taylor & Francis Group, London, 181-193.

Yamashita, K., and Yamada, T. (2009). "Settlement and load sharing of a piled raft with ground improvement on soft ground." *17th ICSMGE, Alexandria, Egypt*, 1236-1239.

Zhang, B., and Small, J. C. (1991). "Analysis of rafts on foundations of variable stiffness." In: G. Beer, J. R. Booker, and J. P. Carter, ed., *7th International Conf on Computer Methods and Advances in Geomechanics*. Cairns, Australia, 1103-1108.

Zhang, H. H., and small, J. C. (2000). "Analysis of capped pile groups subjected to horizontal and vertical loads. ." *Computers and Geotechnics* 26, 1-21.

Appendix A

Calibration Charts for Monitoring Instruments

1. Calibration charts for LVDTs

Figure A-1 LVDT reading versus displacement (mm) for transducer 1

Figure A-2 LVDT reading versus displacement (mm) for transducer 2

Figure A-3 LVDT reading versus displacement (mm) for transducer 3

Figure A-4 LVDT reading versus displacement (mm) for transducer 4

2. Calibration charts for total stress cells

Figure A-5 Pressure transducer reading verses applied pressure (kPa) for transducer 1

Figure A-6 Pressure transducer reading verses applied pressure (kPa) for transducer 2

Figure A-7 Pressure transducer reading verses applied pressure (kPa) for transducer 3

Figure A-8 Pressure transducer reading verses applied pressure (kPa) for transducer 4

3. Deformation versus square root time curve for kaolin slurry

Figure A-5 Deformation versus square root time curve for kaolin slurry

Appendix B

Triaxial Test Data for Soil Specimens Retrieved After Model Tests

Figure B-1 Typical UU stress-strain curve for specimens retrieved after model unpiled raft tests

Figure B-2 Typical UU stress-strain curve for specimens retrieved after load tests on model piles

Figure B-3 Typical UU stress-strain curve for specimens retrieved after 4-pile piled rafts model tests

Figure B-4 Typical UU stress-strain curve for specimens retrieved after 9-pile piled rafts model tests

Figure B-5 Typical UU stress-strain curve for specimens retrieved after 16-pile piled rafts model tests

Appendix C

Interpretation of Bearing Capacity of Unpiled Raft from Model Test Data

Figure C-1 Bearing capacity by two tangent method of unpiled raft on $c_u = 40$ kPa

Figure C-2 Bearing capacity by $0.1B_R$ method of unpiled raft on $c_u = 40$ kPa

Figure C-3 Bearing capacity by log-log method of unpiled raft on $c_u = 40$ kPa

Figure C-4 Bearing capacity by two tangent method of unpiled raft on $c_u = 50$ kPa

Figure C-5 Bearing capacity by $0.1B_R$ method of unpiled raft on $c_u = 50$ kPa

Figure C-6 Bearing capacity by $0.1B_R$ method of unpiled raft on $c_u = 50$ kPa

Appendix D

Development of Interface Contact Pressure in Model Unpiled Raft Test

1. Applied pressure versus Interface contact pressure response

Figure D-1 Applied central point load (P) versus interface contact pressure (σ_c) at corner points in second load cycle – of unpiled raft on clay with $c_u = 40$ kPa

Figure D-2 Applied central point load (P) versus interface contact pressure (σ_c) at quarter points in second load cycle – of unpiled raft on clay with $c_u = 40$ kPa

Figure D-3 Applied central point load (P) versus interface contact pressure (σ_c) at corners in third load cycle – of unpiled raft on clay with $c_u = 40$ kPa

Figure D-4 Applied central point load (P) versus interface contact pressure (σ_c) at quarter points in third load cycle – of unpiled raft on clay with $c_u = 40$ kPa

2. Interface contact pressure versus displacement response

Figure D-5 Measured interface contact pressure (σ_c) versus raft settlement (δ) response in second load cycle – of unpiled raft on clay with $c_u = 40$ kPa

Figure D-6 Measured interface contact pressure (σ_c) versus raft settlement (δ) response in third load cycle – of unpiled raft on clay with $c_u = 40$ kPa

3. Interface contact pressure versus displacement response for the entire loading process

Figure D-7 Measured interface contact pressure (σ_c) versus raft settlement (δ) response for entire loading at corners of unpiled raft on clay with $c_u = 40$ kPa

Figure D-8 Measured interface contact pressure (σ_c) versus raft settlement (δ) response for entire loading at quarter points of unpiled raft on clay with $c_u = 40$ kPa

Appendix E

Differential Settlement of Piled Rafts from Model Tests

Figure E-1 Applied central point load (P) versus differential settlement ($\Delta\delta$) of 4-, and 9-pile piled rafts with piles of $B = 20$ mm and $L = 200$ mm

Figure E-2 Applied central point load (P) versus differential settlement ($\Delta\delta$) of 4-, 9- and 16-pile piled rafts with piles of $B = 30$ mm and $L = 200$ mm

Figure E-3 Applied central point load (P) versus differential settlement ($\Delta\delta$) of 4- and 9-pile piled rafts with piles of $B = 20$ mm and $L = 400$ mm

Figure E-4 Applied central point load (P) versus differential settlement ($\Delta\delta$) of 4- and 9-pile piled rafts with piles of $B = 30$ mm and $L = 400$ mm

Appendix F

Mobilisation of Geotechnical Resistance of Piles in Model Tests

Figure F-1 Variation of mobilized geotechnical resistance of individual piles of $B = 20$ mm and $L = 200$ mm in 4- and 9-pile piled rafts

Figure F-2 Variation of mobilized geotechnical resistance of individual piles of $B = 30$ mm and $L = 200$ mm in 4-, 9- and 16-pile piled rafts

Figure F-3 Variation of mobilized geotechnical resistance of individual piles of $B = 20$ mm and $L = 400$ mm in 4- and 9-pile piled rafts

Figure F-4 Variation of mobilized geotechnical resistance of individual piles of $B = 30$ mm and $L = 400$ mm in 4- and 9-pile piled rafts

Appendix G

Development of Interface Contact Pressure in Model Piled Raft Tests

Figure G-1 Mobilisation of interface contact pressure with applied load at quarter points in 4- and 9-pile piled rafts with $B = 20$ mm and $L = 200$ mm

Figure G-2 Mobilisation of interface contact pressure with applied load at quarter points in 4-, 9- and 16-pile piled rafts with $B = 30$ mm and $L = 200$ mm

Figure G-3 Mobilisation of interface contact pressure with applied load at quarter points in 4- and 9-pile piled rafts with $B = 20$ mm and $L = 400$ mm

Figure G-4 Mobilisation of interface contact pressure with applied load at quarter points in 4- and 9-pile piled rafts with $B = 30$ mm and $L = 400$ mm

Figure G-5 Mobilisation of interface contact pressure with applied load at corner in 4- and 9-pile piled rafts with $B = 20$ mm and $L = 200$ mm

Figure G-6 Mobilisation of interface contact pressure with applied load at corner in 4-, 9- and 16-pile piled rafts with $B = 30$ mm and $L = 200$ mm

Figure G-7 Mobilisation of interface contact pressure with applied load at corner in 4- and 9-pile piled rafts with $B = 20$ mm and $L = 400$ mm

Figure G-8 Mobilisation of interface contact pressure with applied load at corner in 4- and 9-pile piled rafts with $B = 30$ mm and $L = 400$ mm

Appendix H

Interpretation of Bearing Capacity of Model Piled Rafts

1. From model tests

Figure H-1 Representative single stage applied load (P) versus average settlement (δ_{avg}) curve for 4PB2L2 and interpretation of bearing capacity

Figure H-2 Representative single stage applied load (P) versus average settlement (δ_{avg}) curve for 4PB3L2 and interpretation of bearing capacity

Figure H-3 Representative single stage applied load (P) versus average settlement (δ_{avg}) curve for 4PB2L4 and interpretation of bearing capacity

Figure H-4 Representative single stage applied load (P) versus average settlement (δ_{avg}) curve for 4PB3L4 and interpretation of bearing capacity

Figure H-5 Representative single stage applied load (P) versus average settlement (δ_{avg}) curve for 9PB2L2 and interpretation of bearing capacity

Figure H-6 Representative single stage applied load (P) versus average settlement (δ_{avg}) curve for 9PB3L2 and interpretation of bearing capacity

Figure H-7 Representative single stage applied load (P) versus average settlement (δ_{avg}) curve for 9PB2L4 and interpretation of bearing capacity

Figure H-8 Representative single stage applied load (P) versus average settlement (δ_{avg}) curve for 9PB3L4 and interpretation of bearing capacity

Figure H-9 Representative single stage applied load (P) versus average settlement (δ_{avg}) curve for 16PB3L2 and interpretation of bearing capacity

2. From FE analysis

Figure H-10 Numerical applied load (P) versus average settlement (δ_{avg}) curve for 4PB2L2 and interpretation of bearing capacity

Figure H-11 Numerical applied load (P) versus average settlement (δ_{avg}) curve for 4PB3L2 and interpretation of bearing capacity

Figure H-12 Numerical applied load (P) versus average settlement (δ_{avg}) curve for 4PB2L4 and interpretation of bearing capacity

Figure H-13 Numerical applied load (P) versus average settlement (δ_{avg}) curve for 4PB3L4 and interpretation of bearing capacity

Figure H-14 Numerical applied load (P) versus average settlement (δ_{avg}) curve for 9PB2L2 and interpretation of bearing capacity

Figure H-16 Numerical applied load (P) versus average settlement (δ_{avg}) curve for 9PB3L2 and interpretation of bearing capacity

Figure H-15 Numerical applied load (P) versus average settlement (δ_{avg}) curve for 9PB2L4 and interpretation of bearing capacity

Figure H-17 Numerical applied load (P) versus average settlement (δ_{avg}) curve for 9PB3L4 and interpretation of bearing capacity

Figure H-18 Numerical applied load (P) versus average settlement (δ_{avg}) curve for 16PB3L2 and interpretation of bearing capacity