FOUNDATION DESIGN FOR THE MARINA BARRAGE, SINGAPORE

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1 INTRODUCTION

Singapore's Marina Barrage is a major government project to provide a multi-purpose urban reservoir integrating flood control, water supply and lifestyle attractions in the heart of the city of Singapore. The SGD$226 million project includes the construction of a 305m long barrage at the mouth of the Marina Channel to form a freshwater reservoir and will play a pivotal role in enhancing Singapore's water supply.

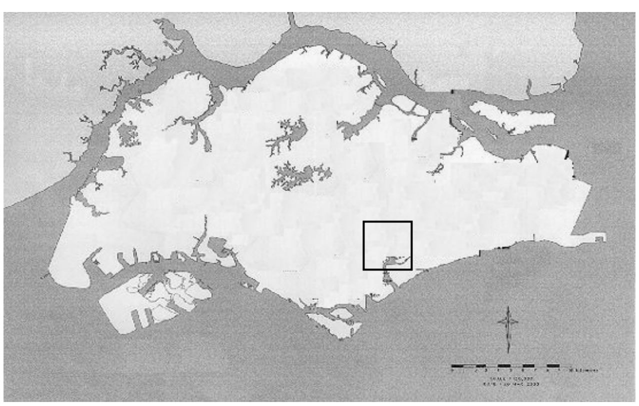
This paper describes how the engineering challenges for the geotechnical aspects of this project were addressed, including the detailed geotechnical analyses and design of the Barrage piled foundation, comprising over 275 bored piles of diameters ranging from 1 m to 1.5 m. With such a large number of piles of different diameters, the challenge was to simulate the pile group interaction effects under both lateral and vertical loading (including negative skin friction due to the consolidating soft Marine Clay deposits) in order to optimise the pile group design.

2 PROJECT OVERVIEW

Singapore's demand for water has grown nine times since the 1950s to about 300 million gallons a day and is estimated to increase by another 20% in 10 years' time. Rain has been the traditional source of water in Singapore for many years, but with limited land and an ever growing demand, new water sources need to be found. The construction of the Marina Barrage and the formation of a fresh water reservoir will provide such a new source.

When the Marina Barrage is completed in late 2008, it will form Singapore's fifteenth reservoir and the first in the city, a fact that makes the project unique. The Marina Reservoir will have the largest and most urbanised catchment at 10,000 hectares, or one-sixth of Singapore. In addition, the Barrage will also act as a tidal barrier and alleviate flooding in low-lying areas such as Chinatown, Boat Quay, Jalan Basar and Geylang. As the water in the Marina Reservoir will be unaffected by the tides, the Barrage will also provide a unique leisure and recreational facility for the Singaporean community.

The Marina Barrage consists of nine 26.8 m long hydraulically operated crest gates which, under normal conditions, will remain closed to isolate the reservoir from the sea. During extreme storm events when the tide is low, they will be lowered to release excess flows from the reservoir. When the tide is high, a large capacity pumping station will operate, maintaining a constant water level within the reservoir. The location and schematic layout of the Marina Barrage is shown in Figure 1.



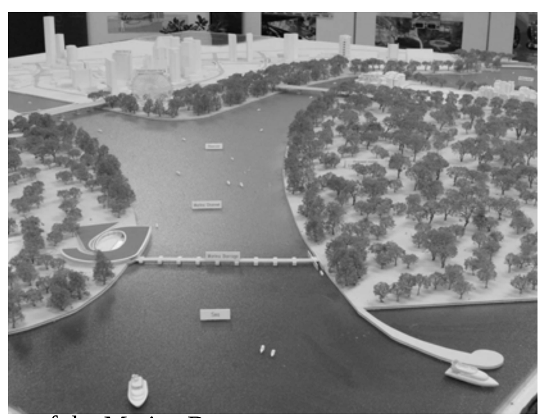


Figure 1: Location and Schematic Layout of the Marina Barrage.

3 GEOLOGY

3.1 REGIONAL GEOLOGY

The site is located in the southern part of the main island of Singapore and the geology of the Marina Bay area is understood to typically comprise a sequence of unconsolidated to normally consolidated recent marine through to fluvial sediments of the Kallang Formation, deposited in the Holocene period (i.e. up to about 10,000 years ago). The Kallang Formation materials comprise very soft to firm cohesive soils and very loose to medium dense granular soils. These materials overlie the Old Alluvium, which consists of stiff to hard cohesive soils or weakly to moderately cemented medium dense to very dense granular soils and was deposited during the Pleistocene period. The land based areas of the site have been reclaimed, where fill has been placed over the *in situ* strata. The history of the reclamation construction is unknown, however it is assumed to have occurred in the last 20 to 30 years.

3.2 LOCAL GEOLOGY AND CLASSIFICATION NOMENCLATURE

A locally developed material classification system based on both geological origin and engineering properties is widely used in ground engineering in Singapore and has been used for this project. The material types and codes (with brief descriptions) are given in Table 1.

Table 1: Geological Classification System

Material	Type	Code	Typical materials	Formation
SOIL	Fill	Fill	Loose to medium dense sand, gravelly sand and silty sand	
	Beach	B	Sand, sometimes silty with gravel, coral and shells	KALLANG (Littoral)
	Estuarine	E	Peat, peaty and organic clays	KALLANG (Transitional)
	Fluvial	F1	Predominantly granular soils including silty sands, clayey sands and sandy silts	KALLANG (Alluvial)
		F2	Cohesive soils including silty clays, sandy clays and clayey silts	
	Marine	M	Very soft to soft, blue or grey clay	KALLANG (Marine Member)
	Old Alluvium	O	Medium dense to very dense silty sand, and firm to hard silty clay	OLD ALLUVIUM

The general geology of the Marina Barrage site comprises:

- Fill materials (Fill)
- Reclaimed land fill (Fill)
- Alluvial Deposits derived from fluvial sources (F1 and F2)
- Marine Clay (M)
- Old Alluvium (OA)

The characteristics of the various geological units encountered are described in the following sections.

Reclamation Fill

Reclamation fill has been placed in the land based areas of the site and the nature of this fill varies depending on the source materials used in the reclamation works. From the findings of the boreholes drilled on land, the reclamation fill is typically described as loose to medium dense sand, gravelly sand and silty sand between 11.5 m and 26.8 m thick.

3.2.1 Kallang Formation

The Kallang Formation is the formation of geologically recent sediments close to the surface and covers about 25% of the land area in Singapore. The formation is named from the Kallang River Basin in the south part of Singapore where it is most extensive. The deposits are generally low lying and are seldom recognised more than 4 m above sea level. The formation commonly overlies the eroded upper surface of the Old Alluvium formation; hence rapid changes in thickness of the stratum are common.

The overlying young sediments of marine, alluvial, littoral and estuarine origin are generally loose or soft in consistency. Five members are recognised within the formation, being:

- Transitional (E)
- Alluvial (F1, F2)
- Marine (M)
- Reef
- Littoral

The Alluvial and Marine Members were encountered in the boreholes drilled for the Marina Barrage project and a description of these two units is given below.

Alluvial Member (F1, F2)

The Alluvial Member comprises Fluvial Sand (F1) and Fluvial Clay (F2) and is found as valley fill throughout Singapore and varies from pebble beds through sand, silty sand and clay to peat. The F1 materials are predominantly loose to medium dense clayey or silty sand whereas the F2 materials are typically firm sandy or silty clay. These two deposits are commonly interbedded and appear to grade laterally into one another. Both deposits locally contain organic lenses.

Marine Member (M)

The Marine Member of the Kallang Formation forms a soft clay blanket over much of the offshore zone of Singapore Island. The thickness of the unit is highly variable. Where thicker sequences occur, the Marine Member comprises an Upper [M(Upper)] and Lower [M(Lower)] division, which have been laid down in different times and are separated by an unconformity usually marked by a weathered crust on top of the Lower Member where the clay is much stiffer, or by sandy fluvial, littoral or organic soils[M(Desiccated)]. The Lower Member was deposited unconformably over valley and plain floors and over fluvial sediments.

Though differing in age, both Members share similarities in appearance and both are essentially an unconsolidated to normally consolidated, kaolinite-rich, pale grey to dark blue-grey, silty clay with sandy, silty, peaty and shelly layers and shell fragments, frequently exhibiting a very soft to soft consistency in their natural state. The Marine Clay varies both laterally and vertically and a maximum overall thickness of 35 m has been recorded in Singapore.

3.2.2 Old Alluvium (OA)

Geologically, the Old Alluvium is thought to represent a deltaic deposit, which was laid down in a slowly subsiding basin during one of the Pleistocene interglacial periods. The Old Alluvium covers virtually the whole eastern section of Singapore Island and has a recorded maximum thickness in excess of 185 m. The deposit is recorded in published literature as thickly bedded, however bedding planes and joints are seldom encountered. The soil texture varies from slightly clayey sand through sandy and clayey silt to silty clay with the majority soil type being medium dense to very dense silty to clayey sand. Bands of coarse sub-angular and sub-rounded gravel can locally occur with a maximum recorded clast size of 60 mm. Clasts are usually composed of quartz, quartzite or feldspar.

Due to its depositional environment, the Old Alluvium is laterally and vertically highly variable with rapid and frequent variations leading to difficulty in correlation of individual lithological horizons. There is however evidence that beds of finer grained material are more common at depth.

In geotechnical terms, the Old Alluvium is considered to be over-consolidated. Debate exists as to whether the material possesses cohesion in the drained state. A number of authors refer to partial cementation of the Old Alluvium as being mechanism for cohesion but they also draw attention to the lateral impersistence of this characteristic. The strength and deformation properties of the Old Alluvium vary over a wide range. Various engineering subdivisions have been put forward by a range of authors based on SPT-N value. CP4:2003 gives recommendations regarding the categorisation of the Old Alluvium in terms of SPT-N and weathering pattern as shown in Table 2 and so these categories have been applied where appropriate.

Table 2: Weathering Classification of Old Alluvium (CP4:2003)

Class	Classifier	Characteristics	Indicative SPT N-value
A	Unweathered	Original Strength	>50 (cannot usually be penetrated by cone penetrometer (CPT) with 20T pushing capacity)
B	Partially Weathered	Slightly reduced strength	>50 (cannot usually be penetrated by CPTs with 20T pushing capacity)
C	Distinctly Weathered	Further weakened	30 to 50
D	Destructured	Greatly weakened, often mottled, bedding disturbance	10 to 30
E	Residual	Bedding destroyed	<10

4 APPROACH TO THE FOUNDATION DESIGN PROCESS

The following process was undertaken in the foundation design for the Barrage structure:

- Geotechnical site characterization, including site visits, as appropriate, during the site investigation process;
- Development of geotechnical models for various parts of the site;
- Assessment of foundation requirements for ultimate limit state (bearing capacity, overall stability);
- Assessment of foundation settlements, differential settlements and lateral movements;
- Assessment of effects of cyclic loading on foundation capacity and deformations;
- Assessment of loads and bending moments required for structural design of the foundation elements;
- Consideration of the effects of down drag due to on-going consolidation of compressible sediments, dewatering, excavation and other construction activities;
- Evaluation of load test data and modification, if necessary, of foundation design parameters;
- Consideration of the effects of construction on adjacent sites, or on other facilities within this site;
- Evaluation of measured performance in relation to predicted performance.

This approach was adopted for the foundation design of the Barrage, which will be subjected to vertical compression loads, vertical tension loads and lateral loads due to the construction and operation of the Barrage, together with additional loads resulting from the ongoing consolidation settlement of the soft Kallang Formation deposits and cyclic loading due to wave action.

The detailed design of the foundations was undertaken in accordance with local codes of practice (CP4:2003 – Code of Practice for Foundations), which adopt a working stress design approach with factors of safety applied to assess ultimate and serviceability limit state requirements. The foundations for the Barrage structure comprised bored piles of 1.0 m, 1.2 m and 1.5 m diameter. A schematic layout of the piles is presented in Figure 2.

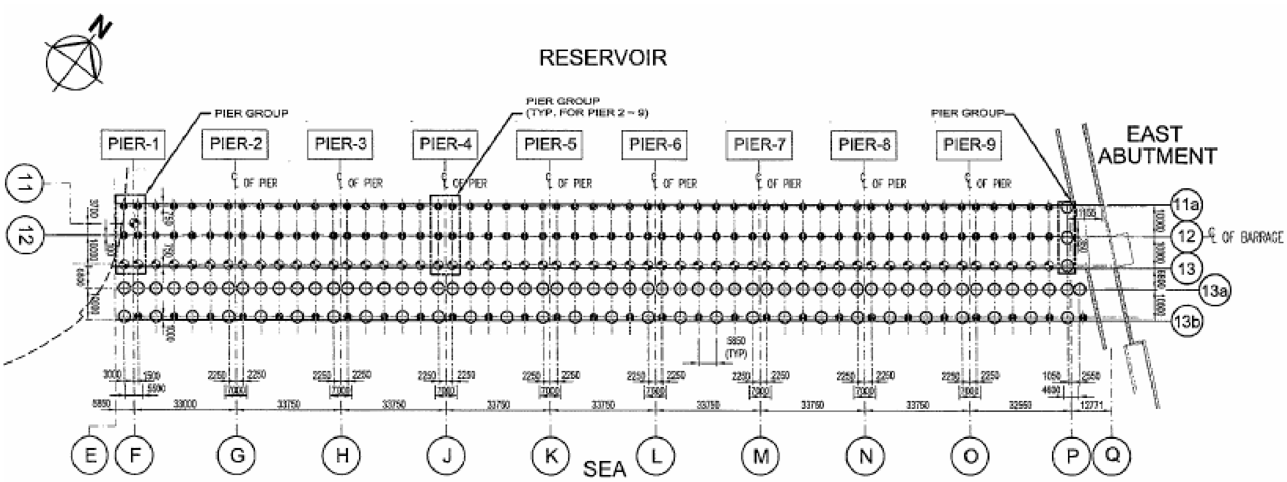


Figure 2: Piling Layout for the Marina Barrage.

Given the nature and thickness of the Kallang Formation, a critical aspect of the foundation design was the performance of the pile group under lateral loading conditions and the ability of the foundation to meet the deflection criteria under serviceability limit state (SLS) loading conditions.

5 CHARACTERISING THE GROUND CONDITIONS

Geotechnical uncertainty is the greatest risk in any deep foundation design and construction process. An accurate knowledge of the ground conditions and reliable prediction of how the ground will behave under foundation loading is essential to reduce risk associated with foundation construction and to develop economical foundation systems which perform to expectations.

A detailed interpretation of the geological and geotechnical conditions based on the client provided land and marine based boreholes was carried in order to:

- Assess anticipated ground conditions for the Marina Barrage piled foundation.
- Develop geotechnical properties and characteristics of the various strata.
- Develop geotechnical design parameters.

A geotechnical data management system (GDMS) was established for all the field and laboratory information available, which provided the opportunity to extend the interpretative possibilities by allowing the increased use of the data in its spatial context. The GDMS provided the Project team with an effective tool to validate the ground investigation data and centrally store the data for easy access. A typical section developed along the alignment of the Barrage structure is presented in Figure 3.

Due to its depositional environment, the Old Alluvium is laterally and vertically highly variable with rapid and frequent variations leading to difficulty in correlation of individual lithological horizons. The strength and deformation properties of the Old Alluvium vary over a wide range, therefore particular care had to be paid in developing the geological model for the Old Alluvium and in selecting the design parameters for the various weathering classes.

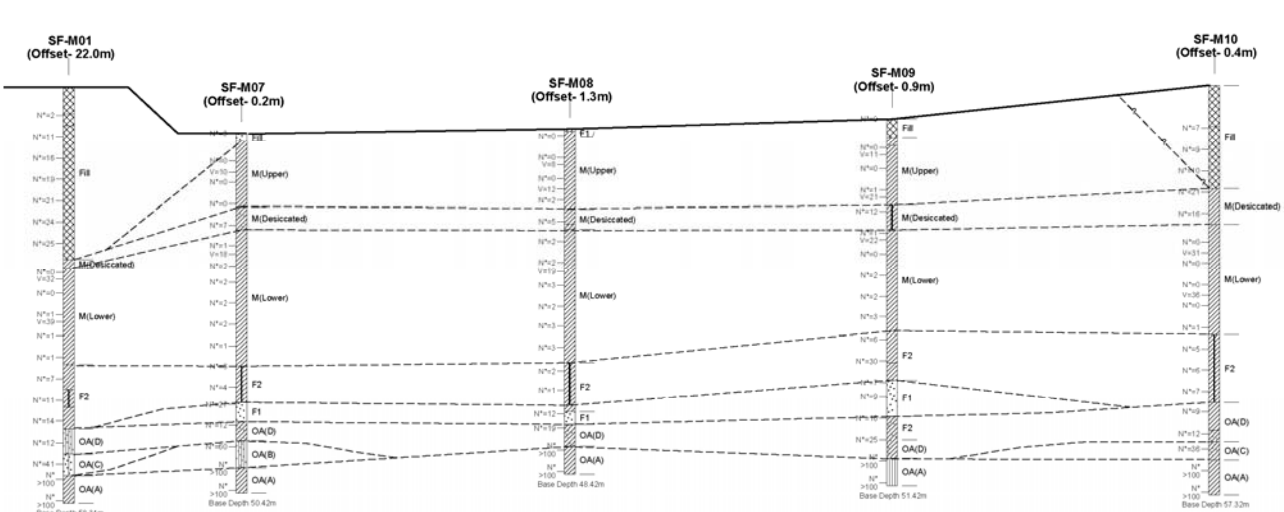


Figure 3: Typical Geological Section along Barrage Alignment.

Using the GDMS, a contour plot of the proposed founding stratum (the unweathered Old Alluvium) for the Barrage piles was prepared, which enabled a more accurate assessment of the founding depth for each pile. The effect of this variation in depth could then be captured in the detailed numerical analyses.

The Project team's experience of both pile foundation design and the local geotechnical conditions meant that we had an excellent appreciation of the critical design issues and were therefore able to focus on the selection of the governing parameters for various loading scenarios (such as those for the soft Marine clay deposits under lateral loading). This approach resulted in the adoption of design parameters that reflected the anticipated ground response without the need to be overly conservative in their selection. Table 3 presents the material parameters adopted in the foundation design.

Table 3: Material Parameters Adopted in Foundation Design

Material	**γ_b (kN/m^3)**	**Ave N**	**c_u (kPa)**	**ϕ_u (Deg)**	**c' (kPa)**	**ϕ' (Deg)**	**E_u (MPa)**	**E' (MPa)**	**ν_u**	**ν'**
Fill	20	10	-	-	0	30	-	10	-	0.3
F1	19	12	-	-	0	32		10	-	0.3
F2	18	7	40	0	0	22	14	12	0.5	0.3
M(Upper)	17	1	$0.25\sigma_v'$	0	0	22	$0.2c_u$	$0.175c_u$	0.5	0.3
M(Desiccated)	20	10	60	0	0	22	12	10	0.5	0.3
M(Lower)	17	1	$0.25\sigma_v'$	0	0	22	$0.2c_u$	$0.175c_u$	0.5	0.3
OA(Class D)	20	20	100	0	0	32	35	30	0.5	0.3
OA(Class C)	21	40	210	0	5	33	90	80	0.5	0.3
OA(Class B)	21	50	270	5	10	35	145	125	0.5	0.3
OA(Class A)	21	100	540	5	20	35	290	250	0.5	0.3

Legend:

γ_b = unit weight; c_u = undrained shear strength; c' = drained cohesion

Ave N = average SPT N value; ϕ_u = undrained friction angle; ϕ' = drained friction angle

E_u = Undrained Modulus (Vertical); E' = Drained Modulus (Vertical); σ_v' = effective vertical stress

ν_u = Undrained Poisson's ratio; ν' = Drained Poisson's ratio

The relationship between undrained shear strength and N value that is widely used for Singapore soils is c_u = 6N (kPa). Orihara & Khoo (1998) proposed the lower bound correlation of c_u = 4N (kPa) for the Old Alluvium. Wong *et al.* (2001) proposed the correlation of c_u = 5.4N (kPa) and it is this latter correlation that was adopted for assessing the design values for the Old Alluvium.

Laboratory test results for undisturbed samples of the Marine Clay material, together with the *in situ* shear vane testing indicate a general increase in strength with depth, therefore a correlation with effective vertical stress was adopted.

The design elastic modulus for the various materials have been assessed using the following published correlations with SPT N values and undrained shear strength, together with values presented in published literature.

Marine (M) members (after Wong *et al.* (1997))

$$E_u = 0.2c_u \text{ (MPa)} \qquad \textbf{(Equation 1)}$$

Kallang Formation F2 material (after Shirlaw (1998))

$$E_u = 2.0N \text{ (MPa)} \qquad \textbf{(Equation 2)}$$

Old Alluvium (based on recommendations given in CIRIA Report 143)

Class D $\quad E_u = 1.8N$ (MPa)

Class C $\quad E_u = 2.3N$ (MPa)

Class B $\quad E_u = 2.9N$ (MPa)

Class A $\quad E_u = 2.9N$ (MPa) **(Equation 3)**

An E'/E_u ratio of 0.87 has been adopted. Also, the horizontal elastic modulus values were taken to be 0.75 of the vertical elastic modulus values.

In total, five geotechnical models were developed for use in the design and were considered to represent the variation in the ground conditions within the Project area. The inferred geotechnical models and pile cut-off levels adopted for design of the piles located along each gridline of the Barrage are summarised in Table 4.

Table 4: Summary of Design Geotechnical Models and Pile Cut-Off Levels

	Gridline 11a	**Gridline 12**	**Gridline 13**	**Gridlines 13b & 13c**
Pile Diameter (m)	1.0	1.0	1.2	1.5 and 1.0
Pile Cut-off Level (m RL)	96.375	94.475	94.275	90.875
Thickness of Sand Fill (m)	4	4	4	4
Thickness of M(Upper) (m)	11.9	10.00	9.3	5.9
Thickness of M(Lower) (m)	18.0	18.0	18.0	18.0
Thickness of F2 (m)	6.0	6.0	6.0	6.0
Thickness of F1(m)	5.0	5.0	3.5	3.5
Thickness of OA Class D (m)	6.0	6.0	7.5	7.5
Thickness of OA Class A (m)	>20.0	>20.0	>20.0	>20.0

6 FOUNDATION DESIGN

6.1 AXIAL CAPACITY

The first stage of the foundation design process was the assessment of the bored pile capacity under vertical compression loading. The bored piles founded on the unweathered Old Alluvium will derive their vertical capacity predominantly by mobilised shaft friction and end bearing in the Old Alluvium.

Careful consideration had to be given to the effects of negative skin friction (NSF) on the piles within the consolidating compressible Kallang Formation marine clay. It has long been recognised that pile foundations located within a settling soil layer will be subjected to negative friction stresses caused by the downward movement of the soil relative to the pile. As a consequence, an additional down drag force will be developed within the pile and the pile head will experience additional settlement. Given the relatively thick layer of marine clay at the site (over 20 m), the effects of down drag on the piles are significant.

The NSF was assessed in accordance with CP4:2003, using the total stress method (α-method) for the cohesive soil. A value of 1 was adopted for the α coefficient and a degree of mobilisation (η) of 1.0 was adopted. Average undrained shear strength values for the M(Upper) and M(Lower) soils were used in the calculation of NSF. As recommended in CP4:2003 for end bearing piles, the neutral plane of the pile was assumed to be at the base of the consolidating soil [i.e. at the base of M(Lower)].

The allowable working load for varying pile diameters was assessed using the inferred geotechnical models, design geotechnical parameters and the proposed cut-off levels for the piles located along each gridline of the Barrage structure. This was carried out using an in-house developed spreadsheet, which produced a series of design charts of pile working load versus pile toe level. These design charts could then be readily used to establish the required pile toe level at each pile location, depending on the prevailing ground conditions at the location, the proposed pile diameter and the required axial capacity.

6.2 PILE HEAD STIFFNESS

Critical input parameters for the 3-dimensional structural numerical analyses undertaken for the Barrage structure were the bored pile head stiffness values for the piled foundation. An assessment of the pile head stiffness was undertaken, which included the effects of pile group interaction, using the computer program DEFPIG, developed by the University of Sydney (Poulos, 1990).

DEFPIG determines the deformations and load distribution within a group of piles subjected to vertical, horizontal and lateral loading. The program incorporates the effect of the pile-soil-pile interaction on individual pile behaviour. A group of 20 piles (i.e. 4 rows of 5 piles) were analysed, which represents the pile arrangement for a typical Barrage pier plus the adjacent row of piles either side of the pier to take account of pile group interaction effects. For the assessment of rotational pile head stiffness, DEFPIG analyses were carried out for a single row of 5 piles with a moment load applied transversely to the pile group.

DEFPIG analyses were carried out for a group of 1000mm, 1200 mm and 1500 mm diameter bored piles for a range of cut-off levels and geotechnical models that represented the variation in both construction and ground conditions along the

Barrage structure. Vertical, horizontal and rotational pile head stiffness values were calculated from the results of the DEFPIG analysis for inclusion in the 3-dimensional structural analysis for the Barrage structure.

6.3 ASSESSMENT OF PILE GROUP DEFLECTIONS

One of the critical performance criteria of the Barrage structure was the magnitude of pile group deflections under construction and operation. It was therefore recognised by the Design team that the deflections obtained from the 3-dimensional structural analysis needed to be verified using independent methods of analyses. This was particularly the case as the structural analysis adopts a rather simplified approach of modelling the piles as discrete linear elastic springs with no interaction effects, whereas in reality the piles will not have a linear response to loading and will interact due to pile-soil-pile interaction because they are founded in a soil continuum.

One of the limitations of the commercially available program DEFPIG is that it can only handle pile groups comprising uniform pile length and diameter. Therefore additional analyses were undertaken to assess pile group deflections using the in-house developed computer program DAMPIG (Poulos, 1997) which is an extension of the program DEFPIG. DAMPIG analyses the response of a pile group with varying axial and lateral pile head stiffness and pile capacities to simulate different pile diameters and lengths, to axial lateral and moment loading.

Again, a group of 20 piles was considered in the analysis (i.e. 4 rows of 5 piles), which represents the pile arrangement for a typical Barrage pier plus the adjacent row of piles either side of the pier to take account of pile group interaction effects. DAMPIG analyses were carried out for a group of 20 piles comprising 1000 mm, 1200 mm and 1500 mm diameter bored piles. For each pile within the group the proposed cut-off level, pile length and appropriate geotechnical model was taken into account in the DAMPIG analysis so that the effect of variations in pile length, diameter and ground conditions were simulated in the analysis.

The results of the DAMPIG analysis indicated maximum vertical displacements of the pile group of about 6 mm and horizontal displacements of about 17 mm. These values compared well with the results from the 3-dimensional structural analysis, giving confidence to the Design team that the overall foundation behaviour was being reasonably captured by the structural analysis.

6.4 ASSESSMENT OF REQUIRED PILE LENGTHS

The assessment of the required pile lengths for the Barrage foundation was based on the calculated vertical pile working loads from the 3-dimensional structural analysis of the Barrage structure using the pile head stiffness values computed by the Geotechnical team.

6.5 REVIEW OF ULTIMATE PILE LOAD TESTS

As part of the design process, a large-scale static ultimate pile load test was carried out on a prototype pile to confirm the assumptions made in the detailed design. The pile was 1000 mm diameter with a total length of 55.8 m below existing ground level. The pile hole was formed using a rotary auger and vibrating wire strain gauges/sensors were fixed to the reinforcing steel cage. Three loading cycles up to a maximum load of about 2.5 times the working load were carried out. The load transfer data obtained from the instrumented test pile was interpreted and provided estimated ultimate skin friction and end bearing values. These values are summarised in Table 5. The measured relationship between end bearing resistance and pile load strongly indicated that the full end bearing resistance had not been fully mobilised during the test.

The in-house developed computer program PIES was used to predict the expected load-settlement behaviour of the test pile, which was compared to the measured load-settlement performance of the pile (refer Figure 4). The recorded pile head settlement reached only about 4% to 5% of the shaft diameter at the end of the test, and there was clearly additional load capacity. If the load-settlement curve were extrapolated to a settlement of about 10% of the diameter (generally accepted as being failure), the geotechnical load capacity would be in the range of 18 to 20 MN, depending on the basis of extrapolation, and well in excess of 2.5 times the working load of 5.57 MN.

The measured vertical pile head stiffness value under the working load was about 970 MN/m compared to the predicted value of about 560 MN/m. The results indicated a pile load-settlement performance for the test pile which exceeded expectation, giving confidence in the pile design parameters and assumptions made in the detailed design.

Table 5: Summary of Results from Pile Load Test

Depth Range (m)	Main Material Type(s)	Parameter	Measured Value	Coffey Design Value	Remarks
0-50.8	Various loose & soft layers (Marine Deposits)	Skin friction	53 kPa	30 kPa (average)	Top 12m cased. Fully mobilized
50.8-52.3	Loose silty sand (OA(D))	Skin friction	99 kPa	40 kPa	Fully mobilized
52.3-53.8	V. hard silty clay (OA(A))	Skin friction	93 kPa	250 kPa	Possibly not fully mobilized
53.8-55.3	V. hard silty clay (OA(A))	Skin friction	324 kPa	250 kPa	Not fully mobilized
> 55.3	V. hard silty clay (OA(A))	End bearing	2.88 MPa	10 MPa	Not fully mobilized

Notes:
OA(D) = Old Alluvium (Class D)
OA(A) = Old Alluvium (Class A)

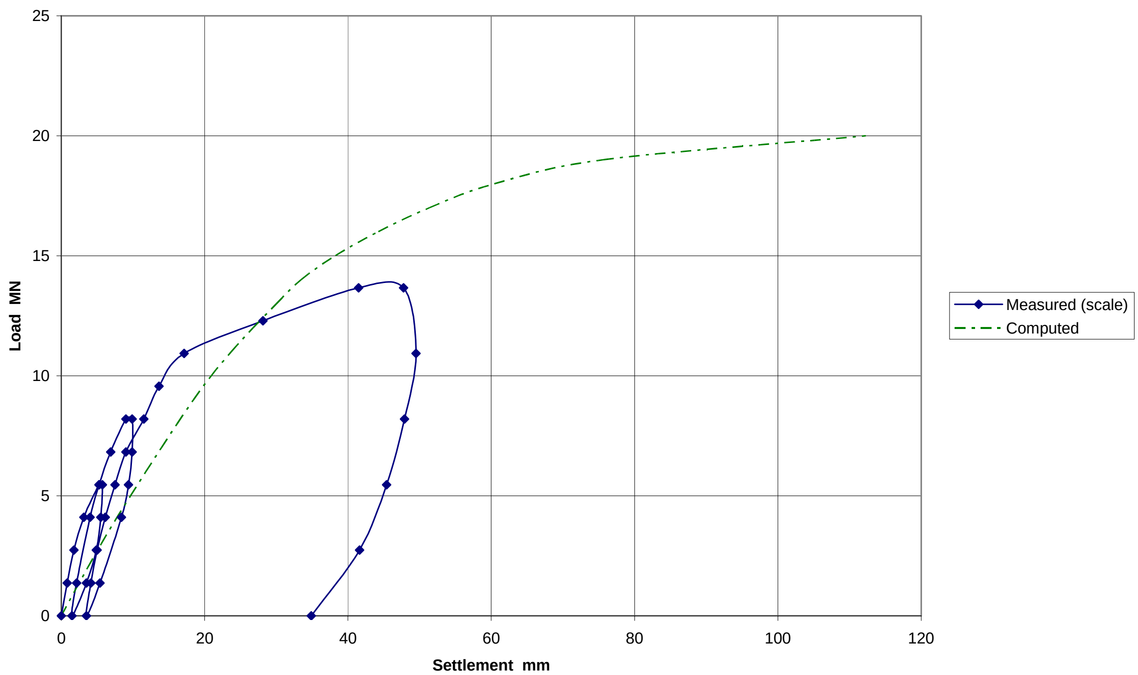


Figure 4: Comparison between measured and predicted load-settlement behaviour of test pile

7 CONCLUSIONS

From a geotechnical perspective, the challenge of the Barrage foundation design was to accurately model the behaviour of the large pile group supporting the 305 m long structure so that an economic foundation solution could be achieved that met performance expectations. The modelling was further complicated by the varying pile diameters and lengths of the pile group as well as the varying ground conditions present along the breadth and length of the Barrage. Working with the Project team in a collaborative manner, the Geotechnical team was able to draw upon its depth of experience of local ground conditions together with its considerable pile design and modelling capabilities to develop and optimise the foundation design.

The Geotechnical team's innovative approach to geotechnical design using its suite of in-house developed computer programs allowed the detailed analysis of the pile group so that factors including pile-soil-pile interaction effects, varying pile diameters and lengths and varying ground conditions within the foundation could be accounted for in the design of this unique Barrage structure. The final piled foundation solution for the Barrage structure resulted in a significant reduction in the required pile lengths and hence resulted in savings in both construction program and costs, to the benefit of both Contractor and Owner.

8 REFERENCES

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