DESIGN OF SHALLOW FOUNDATIONS IN CALCAREOUS SOIL – A CASE STUDY "FREMANTLE ICE WORKS, STAGE 3 DEVELOPMENT"

P.A. BIERMA, BE
Airey Ryan and Hill Consulting Engineers, West Perth, W.A., Australia

Summary

The choice of a structural foundation system depends upon the soil parameters determined from geotechnical site investigation. Recent experience on projects located in Fremantle, approximately 17km from Perth, WA, where the coastal site soils primarily comprise calcareous sand, has demonstrated a lack of correlation between foundation design parameters, determined by traditional geotechnical site investigation techniques, and actual performance. The correlations and methods generally adopted for the determination of foundation design parameters appear to be those used to evaluate siliceous sands. This approach can offer a fair degree of accuracy for silicious sands but does not appear to accurately model calcareous sands. The field investigative techniques, and subsequent interpretation adopted may not always accurately model the degree of cementation present in calcareous soils. When neglected, this can lead to grossly conservative results. A high level of conservatism can result in significant structural foundation costs over and above those which would apply had a more accurate determination of the site soil parameters been made.

On the Fremantle Ice Works Stage 3 site, from the results of Cone Penetrometer Testing (CPT) and inferred soil parameters, the initial recommendation from the investigating geotechnical consulting firm was to install piles on the boundary to accommodate eccentric loading. It was also suggested that consideration be given to piling the entire site to alleviate concerns of liquefaction of the underlying calcareous soils under earthquake loading. Concern was also expressed regarding potential extreme differential settlements of shallow spread footings. Airey Ryan and Hill adopted a raft foundation design unassisted by piles. The modelling of this raft foundation incorporated soil parameters inferred from the initial CPT testing. Subsequent deflection monitoring of the installed raft foundation, during construction, revealed deflections of the order of 2 % of those predicted by Finite Element Analysis and by semi-elastic analysis. This indicates that the inferred design parameters used for modelling the soil strata, particularly the stiffness of the underlying layers, were grossly underestimated. This appears to be due to a serious inability to accurately predict stiffness parameters of calcareous soils using an investigative technique, and parameter inference models, appropriate to other soil types. This is conceivably due to a lack of established local correlations for calcareous soil sites, time and cost constraints associated with the geotechnical investigation, and due consideration given to the highly variable and collapsible nature of calcareous soils.

The adoption of a raft foundation, instead of piles and pile caps, resulted in significant savings for the developer. Given the outstanding performance of the raft foundation, compared to that predicted, the installation of the raft appears with hindsight to also be very conservative. The highly variable nature of calcareous soils, difficulty in accurate modelling of behaviour, and the dire consequences of failure under excess load, typically result in conservative estimates of soil design parameters. This naturally results in conservative foundation design. If a high level of conservatism is adopted by both the investigating geotechnical engineers and the structural engineer responsible for foundation design, the cumulative effect on the cost of foundations can be prohibitive for onshore developments. There is a clear need for cost effective field investigative techniques which can determine calcareous soil design parameters to a reasonable degree of precision. The resultant benefits of cost effective and appropriate foundation design can then be passed onto the developer, and ultimately the consumer.

1 INTRODUCTION

Calcareous sands behave significantly differently to "normal" siliceous sands of similar grain size distribution, and have proven to be troublesome for foundation design, both offshore and onshore. Extensive research of calcareous soils has been performed recently at the University of Western Australia, with particular emphasis on foundation systems for offshore structures. However, comparatively little information exists which is specific to the design and performance of onshore shallow foundations on calcareous soils.

The degree of cementation of calcareous soils impacts significantly on its load bearing capacity. The highly variable degree of cementation of a calcareous soil can vary the soil response to load from, virtually that of a solid rock, to that of a very weak, collapsible, unstable soil.

The common existence of cemented layers of calcareous soil at the surface, with weak underlying layers, has often led to concern of liquefaction of the soil under foundations subjected to cyclic earthquake loads, and the possibility of resultant excessive shallow foundation settlement. In the case of the Fremantle Ice Works Stage 3 development, it has been demonstrated that the CPT testing performed underestimated the stiffness and strength of both the cemented layer near the surface and the underlying weaker material. The bearing pressures applied by a medium sized development are relatively "light" when compared to the bearing pressures likely to be exerted by piles or large storage tanks.

This paper presents a case study of the Fremantle Iceworks Stage 3 development. The gross differences between the calculated foundation deflection based upon parameters inferred from the site investigation work, and the deflection that occurred (which was monitored throughout construction) clearly indicate erroneous soil stiffness values derived from the initial site investigation. The soil stiffness parameters derived from the site soil stratigraphy have been demonstrated to grossly underestimate the degree of cementation of the calcareous site soils and the stiffness of the underlying less well cemented layers.

2 GEOTECHNICAL SITE EVALUATION

The "Iceworks" Stage 3 Development consists of a four storey apartment block on a 30m x 22m site within a kilometre of the coast in Fremantle, WA. The adjoining sites comprise a 3 storey building to the east, a 1 storey car deck to the south and a 2 storey building to the west.

The soil profile typically found below the Fremantle area comprises layers of sand, derived from Tamala Limestone, and/or layers of calcareous soil derived from the Safety Bay geological unit, with varying degrees of cementation, overlying weathered limestone rock at depth.

2.1 Fieldwork

The fieldwork for the geotechnical investigation consisted of 3 CPT tests and water level and height level surveys at each CPT test location.

The investigating geotechnical firm reported that, in general, the results of the CPT tests performed indicated soil of lesser strength than that inferred at other nearby sites in the Fremantle region.

The water table was reported to exist at a depth of between 1.05m and 1.25m below the natural ground level.

The CPT tests typically met refusal at 15 to 16m depth, indicating an underlying layer of very strongly cemented calcareous soil or weathered limestone cap rock.

2.2 Inferred Soil Strata & Parameters

Based on the Geological Map of Western Australia, Fremantle Sheet, and the results of the CPT tests, a generalised subsurface soil profile was prepared including an estimate of the Elastic Modulus (E) for each soil layer.

The CPT test cone resistances recorded appeared to indicate particularly low strength soil layers between depths of approximately 3 to 4m and again at 10 to 15m below the existing ground surface. The tip resistances recorded were of the order of 1.25 MPa and 0.5 MPa respectively.

The relationship between EFCP tip resistance (qc) and Elastic Modulus (E) is typically given by :

$$E = \alpha \, qc \qquad (1)$$

where :

E = Estimated soil layer elastic modulus (MPa)
q_c = EFCP tip resistance (MPa)
α = Constant

The empirical constant (α) relating the soil layer stiffness and EFCP tip resistance is generally dependent on the relative density, effective vertical stress and degree of overconsolidation of soil strata being considered and local experience. For the Iceworks Stage 3 site, the weak layers were assigned a constant of 4.0, and for the other layers the constant was approximately 2.7. The inferred geotechnical model for the Iceworks, Stage 3 site is as per Figure 1 below.

Layer	Depth Range (m)	Consistency	Elastic Mod. (MPa)	Description
1	0 - 3	Medium Dense	20	Sand
2	3 - 4	Loose	5	Sandy Silt
3	4 - 8	Loose/Med. Dense	16	Silty Sand
4	8 - 10	Loose	10	Sandy Silt
5	10 - 15	Very Loose/Soft	1	Clayey Silt
6	> 15	Cemented	200+	Limestone

Figure 1: Inferred geotechnical model.

2.3 Recommended Structural Response

Based on the inferred geotechnical model described in Figure 1, estimates of allowable bearing pressure and settlement for different sized shallow spread footings were given by the investigating geotechnical firm. For strip footings, and 150kPa design pressure, the calculated elastic deflection was in the range 45 to 90mm and for pad footings was 15 to 40 mm. As the footing size increased, the calculated deflection also increased for the same bearing pressure. It was noted, in the geotechnical report, that the calculated footing deflections were elastic only and that creep settlement could add up to further 50% deflection and interaction with adjacent footings a possible additional 15 to 30%.

Due to the close proximity of the external walls to the boundaries, it was anticipated that piles would be required to accommodate the eccentric loading on the boundary footings. If the boundary footings were to be piled, the estimated pile settlement would be of the order of 5mm. Concern over differential settlement between the piled boundary footings and internal footings led to the recommendation that consideration be given to supporting the entire building on piles if the structure couldn't be designed to accommodate the anticipated differential settlements.

The proposed piled footing system could utilise cast in-situ grout injected piles toed into the limestone approximately 15m below the existing ground surface. The allowable design pile end bearing pressure and skin friction was recommended as 2 MPa and 5kPa respectively.

Adopting the pile design figures quoted above, a 350mm diameter pile would have a theoretical working capacity in the order of 275kN, and a 450mm diameter pile approximately 42kN.

Based upon the estimated building load, in excess of 40 piles, on a 3.85m square grid, drilled to a depth of 15m, would be required to support the four storey structure. Due to the relatively close spacing of the piles, a continuous raft would be desirable for construction purposes, rather than individual pile caps which would essentially cover the majority of the site area in any case.

Another supporting argument presented for the adoption of a piled foundation system, was concern that the potential for liquefaction of the low strength subsurface soils under cyclic earthquake loads existed. Based on a simplified procedure by Tosho Iwasaki for saturated uncemented sandy soils, the site was classified as having a "high" risk of liquefaction if shallow spread footings were adopted.

3 ALTERNATIVE STRUCTURAL RESPONSE

3.1 Supporting Arguments

The high commercial impact of a piling scheme versus a non-pile assisted footing, warranted the investigation of an alternative foundation design despite the liquefaction concern raised by the investigating consulting geotechnical firm.

Settlement of foundations, and resistance to shear failure, for silica sands depends on both the size and shape of foundation and on its depth below the surface and the degree of compaction of the sand (relative density).

Shallow spread footings such as strip and pad footings are suitable for soil where a stratum of high relative density and adequate thickness exists near the surface. In the case of calcareous soils, shallow footings may be suitable if a cemented layer exists at or near the surface, capable of supporting the imposed loads.

These foundations offer little resistance to differential movements, which must be limited to sufficiently low levels, or allowance made for movement joints in the design of the superstructure. Footings of this type are normally utilised only where the site soils possess low compressibility, or they are proportioned to limit design bearing pressures (and settlements) to low levels.

For the Fremantle Iceworks Stage 3 development, Airey Ryan and Hill adopted a raft foundation which was unassisted by piles. This resulted in a saving of the order of $70,000 or approximately 55% of the cost of the piled option.

In appropriate circumstances, a raft footing bridges over soil irregularities, and the average settlement does not approach the extreme values of spread footings. A raft foundation will also greatly restrict differential settlements, and at the boundary will act as a continuous strap footing, reducing the soil bearing pressures on the boundary to acceptable levels. This can negate any requirement for piling to accommodate eccentric loading.

If piles were to be used at close centres, then pile caps would cover the majority of the building footprint, and so a raft would not increase material requirements or site preparation excessively. In addition, adaptation of the raft foundation to function as the ground slab for the development was possible, providing further cost savings.

The raft foundation is able to spread the building loads over the largest possible area, and reduce the design bearing pressures on the subsoil strata. A raft foundation appeared appropriate, provided total and differential deflections could be limited to acceptable levels, and the issue of liquefaction of the underlying weaker soil strata under earthquake loading could be addressed.

3.2 Background Information

For normal silica sands the ultimate bearing capacity is dependent on the soil unit weight and angle of internal friction which varies primarily with the relative density of the soil. In some circumstances the soil bearing capacity of a raft foundation is also increased due to the larger foundation size and greater founding depth. Terzaghi and Peck (1948) indicate that an increase in allowable bearing pressure of up to 100% is possible for silica sands.

This situation is however, not directly applicable to calcareous soils which differ significantly from silica sands in geological formation, chemistry, physical properties and performance under load. Calcareous sands generally exist in a very open structure, with an immense scope for variation in properties which are influenced by CaCo3 content, grain size distribution, in situ density and the degree of cementation.

For shallow footings on uncemented calcareous soils the bearing capacity depends primarily on the compressibility of the soil and is largely independent of footing size. This has been demonstrated by centrifuge testing performed at the University of Western Australia by Finnie and Randolph (1) which has shown that the relationship between applied pressure and displacement is almost linear, and that there is no well defined yield point or "ultimate bearing pressure". Instead it is proposed that the 'bearing capacity' of the soil be measured as that which results in acceptable footing settlement for the superstructure being considered. The mode of deformation of the soil is that of localised shearing of the soil particles at the perimeter of the footing. Finnie and Randolph (1) report that previous studies of punching shear failure of un-cemented calcareous soil reveals the lack of a rational approach to predicting the bearing capacity.

Odometer tests, on offshore and onshore uncemented calcareous soils, performed by Joer et.al. (2) indicate that for the onshore deposit, which consists primarily of uniform coarse rounded particles or pellets, there is a quasi linear increase in soil stiffness with controlled displacement. Tests performed on samples with different initial voids ratio indicate a convergence of settlement versus bearing pressure irrespective of the initial voids ratio. This, however, is limited to very high bearing pressures, such as those typical for driven piles, and not those appropriate for shallow foundations. Acceptable settlement limits for shallow foundations correspond to 'low' bearing pressures, and are largely influenced by the initial voids ratio. The work performed by Joer et.al. (2) is confirmed by Finnie and Randolph (1), which indicates that under drained conditions, the major contributing factor towards foundation settlement is compression of the soil directly below the foundation, which is largely independent of foundation size. The primary mode of settlement is compression of the open structure calcareous soil, which is not accompanied by soil yield and displacement that might be expected for denser silica sands.

Soil improvement techniques, such as vibro-compaction can improve calcareous soils to be subjected to 'high' loads likely to break any cementation of soil particles, if present. However, for most onshore sites, where there is some cementation present however weak, application of vibro-compaction will break any cementation between particles, which may not have been broken under 'low' loads. This could result in a reduction in the soil stiffness compared to that available had the bonds between grains not been broken. All that may be achieved is a uniformity in soil stiffness across a particular site which aids design for differential settlement. Vibro techniques for onshore developments, in close proximity to one another, are also not suitable due to possible increased settlement of adjacent structures at the boundary due to a reduction in soil stiffness. In addition damage to adjacent structures can occur due to vibration and compaction of the underlying soil strata.

Where 'low' loads are likely to be imposed on a weakly cemented calcareous soil site it is critical to determine if the imposed bearing pressures are likely to destroy any cementation present, or whether the degree of cementation is likely to reduce in the future, resulting in foundation settlement beyond that anticipated.

A common situation for both onshore and offshore calcareous soil, is that a cemented layer of calcareous soil exists at or near the surface, with weaker layers of un-cemented or variably cemented layers underneath. This is clearly indicated by the CPT test logs for the Fremantle Iceworks site, and is also evident for several other sites in the Fremantle region.

The cemented crust acts to spread the applied shallow foundation loads over a larger region. The crust can in some situations span over weaker subsurface material, shielding it from both overburden pressure and applied load.

Exceeding the strength of the upper cemented crust can lead to a sudden punch through failure accompanied by uncontrolled settlement. Finnie and Randolph (3) observed that the failure mechanism, and resultant settlement, was dependent on both the ratio of the crust thickness to foundation size, and the relative stiffness of the underlying material.

For a 'thin' crust where the thickness ratio of the depth of cemented crust to the foundation size is less than 0.4, the crust acts like a spreader beam and deflects in a bending mode. With increasing load, the flexural tensile capacity of the cemented layer is eventually exceeded and tension cracks initiate at points beneath the perimeter of the foundation and propagate upwards through the crust. Refer to Figure 2 below reproduced from Finnie and Randolph (3).

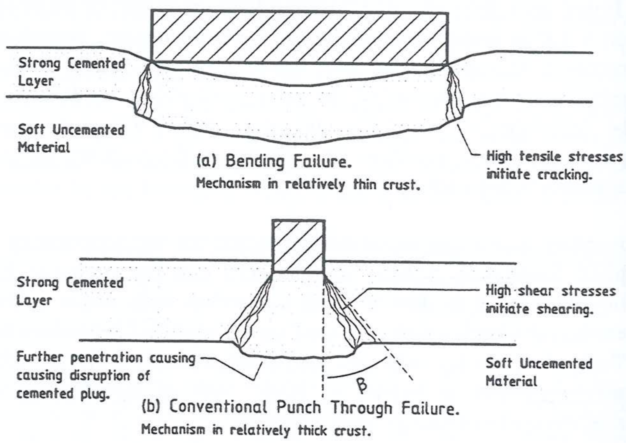


Figure 2: Failure modes for penetration of cemented crust overlying uncemented material.

The degree of settlement is governed by the stiffness of the underlying material and the laboratory results of Finnie and Randolph (3) indicate the bearing modulus is within a factor of 2 of the bearing modulus for the underlying material had no crust been present. This is thought to be due to 'load spread' by the upper cemented layer for small settlements.

Finnie and Randolph (3) claim the degree of load spread is approximately over an area 50% larger than the foundation area which accounts for a bearing modulus of the order of two times greater than had no crust been present.

If the underlying material is uncemented calcareous sand, the bearing modulus is largely independent of foundation size and primarily dependent on the compressibility of the calcareous sand. No strain softening was observed during the testing by Finnie and Randolph (3), with a monotonic increase in bearing capacity with foundation settlement.

For a 'thick' upper crust with a thickness to foundation size ratio in excess of 0.4, the failure mechanism was observed, by Finnie and Randolph (3), to be a punching shear failure through the upper crust in a conical shape approximately 40 degrees to the vertical (refer to Figure 2). The central sheared plug was likened to a stubby pile with crushing of the sheared crust below the foundation possible prior to shearing through to the underlying weaker material with increasing load. For the case where the underlying material was silt, significant strain softening was observed, which could be classified as a sudden punch-through failure with uncontrolled settlement. Where the underlying material was calcareous sand, crushing of the sheared crust 'plug' was also observed but there was no strain softening with punch through to the underlying material.

3.3 Raft Foundation Modelling & Design

Both Finite Element Analysis (FEA) performed by Soil and Rock Engineering using the FEAR program, and a simplified elastic analysis performed by the Author using SpaceGass, were used to predict the settlement of the Iceworks raft foundation. Both analyses assumed the elastic modulus of the soil layers as given in Figure 1 (inferred from the CPT testing), and the same design loading.

3.3.1 Finite Element Analysis

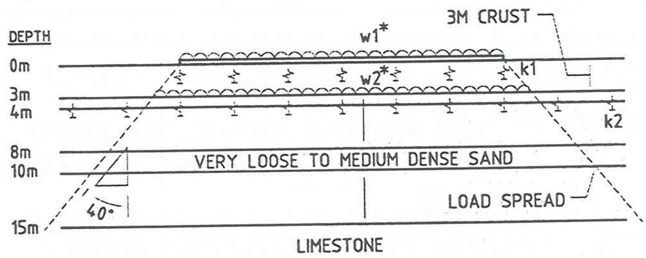
In the case of the FEA performed by an independent geotechnical consulting firm using the FEAR program, the resultant raft curvature could be used to predict the bending moments induced in the raft for reinforcement design. This analysis assumed a uniform loading over the entire surface of the raft and indicated 'dishing' of the raft in the centre. The maximum curvature of the raft was in the short axis direction through the centre of the raft. This indicated a peak ultimate raft bending moment of 130 kNm/m. This is comparable to the moments in the raft calculated assuming an upward uniform bearing pressure exerted by the soil on the base of the raft.

3.3.2 Simplified Elastic Method

Three simplified elastic analyses of the raft were performed by the author. The first comprised a one dimensional model of the estimated wall loads on the 400mm thick concrete raft down the long axis, with the raft supported on springs at 500mm centres. The spring stiffness adopted corresponded to the equivalent soil subgrade modulus taken over the full 15m of soil strata. This analysis indicated an almost uniform soil bearing pressure due to the high stiffness of the raft foundation with respect to the soil strata. This one dimensional analysis indicated maximum settlement at the edges of the raft with hogging between the extreme external walls. This analysis does not model the distortion of the soil surrounding the raft, and is in direct contrast with the 'dishing' indicated by the Finite Element Analysis which also accounts for the two dimensional nature of the raft foundation.

The second two simplified elastic analyses assumed that load spread, due to the stiff upper 3m of soil strata, was occurring as investigated by Finnie and Randolph (3). The Iceworks Stage 3 raft foundation being approximately 20m wide gives a thickness ratio of 0.15 to the 3m stiff crust identified in the CPT test logs. The cone tip resistance for this layer lay in the range from 5 to 15 MPa. A thickness ratio of less than 0.4 indicates a 'bending' type response of the upper layer with load spread at approximately 40 degrees from vertical through the layer. Finnie and Randolph (3) estimated that, based upon CPT tests of prototype foundations on artificially cemented layers subjected to centrifuge testing, cone resistances in the order of 6-10 MPa corresponded to a weakly cemented layer, 10-16 MPa for a medium strength cemented layer and 30-45 MPa for a strongly cemented layer. For the purpose of analysis, the 3.0m stiff layer was assumed to be 'strongly cemented' with a Young's modulus, E of 200MPa and a beam tensile strength of 1.2MPa.

The effect of the loading of the raft foundation on the 3.0m thick upper 'crust', and the spread loading of the raft on the underlying 12m of weaker soil strata was modelled separately. Refer to Figure 3.



w2* < w1* (Approx 70%)
E2 (12m SOIL STRATA) = 3.8 MPa < E1 (3m CRUST) = 20 MPa
k2 < k1 (Approx 4.7%) 3m CRUST VERSUS 12m SOIL STRATA.
S.E. No.1-30m X 22m RAFT LOAD w1* ON 3m CEMENTED CRUST MODELLED BY SPRINGS (k1) AT 500mm CRS.
S.E. No.2-REDUCED RAFT LOAD w2* OVER 150m x 150m OF 12m DEEP SOIL STRATA MODELLED BY 3M DEEP 'SOIL' BEAMS ON SPRINGS (k2) AT 5m CRS.

Figure 3: Simplified elastic analysis model.

The first analysis modelled the raft as being supported by a beam grillage with beams at 5m centres over an area of 150m by 150m in plan. The soil surrounding the raft was modelled as 3m deep by 5m wide 'soil' beams with a Young's Modulus of 200MPa. The raft was similarly modelled as concrete beams 400mm deep at 5m centres with a Young's Modulus of 28,600MPa corresponding to the 25MPa concrete to be used for the foundation. The 'soil' beams and raft were supported by springs of 20MPa stiffness equal to the inferred modulus of the 3m thick upper cemented layer.

The second analysis allowed for load spread through the upper 3m deep layer, and modelled the effect of the reduced raft load on the underlying 12m strata of soil. Again a 'soil' beam grillage was adopted to model the load spread of the upper cemented layer with spring supports of 3.8 Mpa stiffness equal to the equivalent soil modulus for the underlying 12m of soil. The analysis also indicated the greatest curvature of the soil (and raft) through the short axis of the raft through the raft centre. This maximum curvature corresponds to an ultimate (factored) positive bending moment in the 5m wide by 3m deep 'soil' beam of 1439kNm under the centre of the raft, and an ultimate peak negative moment of 823kNm approximately 10m distant from the long axis raft edge.

3.3.3 Theoretical Calculated Deflections

The calculated raft settlements for the different analyses performed are summarised in Figure 4. It is relevant to note that the total raft settlement, for the Simplified Elastic analysis, would be the summation of that calculated for compression of the upper crust (S.E. No.1) and compression of the underlying 12m of soil strata (S.E. No.2).

	Raft Deflection (mm)		
Position	F.E.A.	S.E. No.1	S.E. No.2
Centre	45	3	47
Corner	12	2	23
Midway Long Axis Edge	25	3	35
Midway Short Axis Edge	25	3	30

F.E.A – Finite Element Analysis
S.E. No.1 – Simplified Elastic Analysis of Raft on 3m cemented crust.
S.E. No.2 – Simplified Elastic Analysis of spread Raft load on 3m cemented crust acting on underlying strata.

Figure 4: Calculated raft settlements by Finite Element Analysis and Simplified Elastic Analysis

The typical deflected shape of the 'soil' beam grillage is illustrated by Figure 5. The raft loading, reduced by load spread through the upper 3m crust, is also indicated.

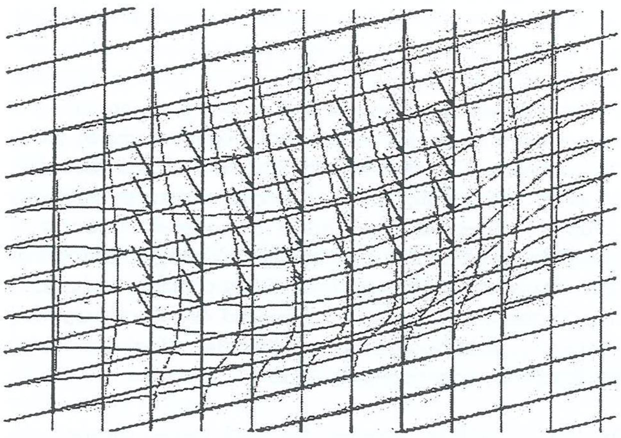


Figure 5: Modelled soil strata deflection

Note that the zone of influence of the raft loading on the underlying soil strata does not extend far beyond the spread raft footprint at the bottom level of the upper crust.

3.3.4 Theoretical Calculated Crust Strength

Based upon the beam flexural tensile strengths quoted in Finnie and Randolph (3) for a 'weak', 'medium' and 'strong' cemented layer pertaining to laboratory prepared cemented crusts, the design moment capacity (unfactored) for a 3m deep by 5m wide soil 'beam' would be 750 kNm, 2250 kNm and 9000 kNm respectively. These capacities indicate a factor of safety against failure of the crust due to the raft load of 1.56 and 6.25 for the 'medium' and 'strong' layers respectively. The crust tensile strengths quoted are however, those of prepared laboratory specimens which have been estimated to correspond to cone resistances of 6–10 MPa for the weakly cemented layer, 10–16 MPa for the medium strength layer and 30–45 MPa for the strongest layer tested. Considering the cone resistance of the upper 3m strata of soil at the Iceworks site is in the order of 5–15 MPa, the theoretical crust flexural capacity is in the order of 1500 kNm. This is within 5% of the calculated ultimate factored design moment acting on the 3m x 5m wide soil 'beam'. More information on the strength and stiffness properties of any such crust at the Iceworks Stage 3 site is required to better determine the crust yield point if in fact a cemented crust exists.

4 FIELD MONITORING VERSUS MODEL

4.1 Raft Deflections

An initial level survey, of the top surface of the raft foundation, was performed on the 1st April, 1998 from a bench mark established at completion of construction of the raft, and far enough away to be outside the raft zone of influence. Further level surveys, performed from the same benchmark, were performed at completion of construction of the first floor, second floor and also several months after project completion. The final set of levels taken on the 2nd December, 1999 indicate a maximum raft settlement of 3mm in the centre, 3mm along the north and south short boundaries, and 1mm midway along both east and west boundary edges. These levels indicate the final raft deflection is of the order of 7% or less than the settlement predicted by both the Finite Element and Simplified Elastic analyses performed on the raft using the soil stiffness moduli inferred from the CPT testing.

4.2 Implications on Assumed Design Parameters

As can be seen, from the raft deflections calculated by the Simplified Elastic analysis, the primary contribution to the total raft settlement is that due to compression of the underlying soil strata, and not compression of the upper cemented crust. The gross differences between the actual raft deflection and that predicted is directly attributable to the inferred soil stiffness modulus quoted for the underlying weaker material.

4.3 Possible Sources of Error

A possible source of error in calculation of the raft settlement may be the lack of modelling of the high degree of confinement due to the adjacent developments to the Iceworks Stage 3 site.

The higher friction ratios of the CPT logs, and drop off in tip resistance just below 2m depth and at 10m depth indicate un-cemented calcareous sand. This approximately relates to the weaker layers, inferred between 3 and 4m depth and between 10 and 15m depth by the investigating geotechnical firm. For the remainder of the logs the friction ratio is relatively low, in the order of 1%, which is characteristic of 'normal' silica sand. The simplified elastic analysis conducted assumes that the settlement or compression of each spring in the model is totally independent to that in adjacent springs. This assumption is valid for calcareous soils where the primary mode of deformation is compression. This is however, not valid for silica sand where the resistance to shear stress and subsequent soil deformation is affected by footing shape, depth and size. Therefore, if a significant portion of the soil strata at the Iceworks Stage 3 site is silica sand, these factors will greatly influence the analysis.

The presence of weak layers in the inferred soil stratigraphy greatly affect the calculated raft settlement due the low E values quoted. Correlations between cone tip resistance (q_c) and Young's Modulus can be misleading for calcareous soils due to the "pressure bulb" created in the soil by the cone during penetration. This pressure bulb can possibly fracture any cementation present in the soil leading to uncharacteristically low tip resistances and hence inferred soil stiffness much lower than that which exists in the undisturbed insitu soil.

Established correlations are generally based on laboratory measured moduli using unaged and reconstituted silica sand samples. This approach does not account for cementation, or past stress or strain history, which can result in significantly higher soil stiffness insitu than anticipated.

5.0 DISCUSSION

5.1 Fieldwork & Soil Design Parameters

The design of shallow foundation systems on sands, particularly calcareous sand sites is generally governed by settlement rather than strength except for unusually high loads or narrow footings.

The cone penetration resistance recorded in the logs is a complex function of both strength and deformation properties of the sand being tested. As no generally applicable analytical solution for soil deformation modulus, as a function of cone tip resistance exists, reliance on established empirical correlations appears to be common practice. However, to the author's knowledge, no such established correlations exist for calcareous soils. This raises the issue of the appropriateness of reliance on silica sand correlations for estimates of soil stiffness for soils anticipated to have a large calcareous sand content. The inference of soil stiffness from CPT test logs is prone to huge variation, even when using well established correlations, and as such relies heavily on local knowledge of site geology. Therefore, in the case of calcareous soil sites, additional testing over CPT testing appears to be required if design parameters are to be determined with any level of accuracy, or good correlations on field data need to be developed.

Insitu tests are preferable, but not always practicable. SPT testing can be useful, due to the well established correlations for number of blows and relative density of soil, or bearing capacity (for silica sands), which may be used for comparison purposes where calcareous soil deposits are suspected. The concept of relative density calcareous soils is not really appropriate due to their high compressibility and crushability of grains. Also, SPT testing can give erroneous results in fine or silty sands of low permeability where there is a drop in pore pressure at the base of the spoon which leads to an increase in effective stress and high blow count readings.

The use of a self boring pressuremeter may be an accurate way of measuring soil stiffness as direct measurements of horizontal displacement versus applied pressure can be made.

Seismic cone testing can be used to verify the stratigraphy highlighted by CPT testing. The speed of shear waves through the soil can be used to estimate the insitu soil elastic modulus without the test destroying any cementation that may be present. Seismic cone tests can also give a good indication of liquefaction potential of calcareous soil sites.

Plate loading tests can be used to give good indications of soil stiffness. These tests are only really appropriate for cemented calcareous soil sites if the plate size and loads are comparable to those to be implemented. Small scale loads and deformations will not indicate any potential punch through failure of a surface cemented crust. Plate loading tests undertaken below the water table can be valuable in measuring pore pressure generation and dissipation for liquefaction analysis.

Laboratory testing, such as the oedometer test, is useful for measuring the one dimensional deformation modulus of silica sands. It is questionable however, whether it is possible to obtain an 'undisturbed' insitu sample of calcareous soil for testing purposes.

The gross difference between the calculated raft settlement and that which actually occurred highlights the need for :

1. The use of supplementary field investigative techniques to confirm / deny the presence of collapsible calcareous soil layers indicated by CPT testing. The use of supplementary field tests, and/or laboratory testing to provide independent estimates of foundation design parameters to those inferred from CPT logs.

2. An industry awareness that use of soil classification charts, and application of high levels of conservatism to correlations applicable to silica sands for calcareous soil deposits, results in safe, but in some instances grossly conservative foundation designs which can be excessively costly to development.

3. Establishment of a database of local experience and correlations for coastal calcareous soil deposits to aid design.

4. A means of determining the presence and properties (Young's Modulus, flexural tensile strength) of any cemented layers of calcareous soil which can be modelled to spread foundation loads and shield weaker underlying layers from imposed loads.

5.2 Liquefaction Potential

The evaluation of the likelihood of soil liquefaction during earthquake loading involves firstly estimating the average cyclic stress ratio likely to be imposed by a design seismic event, and, secondly comparison with the average cyclic stress ratio likely to cause liquefaction of the soil. The current practice in North America, for determination of the cyclic stress ratio to cause liquefaction, is either insitu soil testing and use of field correlations, or laboratory testing on representative soil samples. Because of the difficulty in obtaining undisturbed soil samples, the insitu testing and field correlation approach is generally preferred.

Empiracle correlation charts exist for uncemented sands and silty sands, of cyclic stress ratio versus modified cone tip resistance (from insitu CPT testing) from which the possibility of liquefaction can be evaluated. These are generally applicable to silica sands, in either a fully drained or undrained condition, and must be used in conjunction with local experience.

It has been established that sensitive uncemented fine grained soils of low cone tip resistance and low friction ratio have a high liquefaction potential or liquidity index. For calcareous sands, the high compressibility results in low cone tip resistance, but increased friction ratio. These have a higher resistance to cyclic loading but, the size of the earthquake loading will control their final susceptibility to liquefaction.

Finnie and Randolph (3) have demonstrated that for model tests of shallow foundations on uncemented silt, liquefaction can occur at cyclic load levels as low as 4% of the applied bearing pressure which was 75 kPa. The performance of the soil under cyclic loading was shown to be very sensitive to the local density of the material, with densification of the soil due to preloading by a factor of two, increasing the required liquefaction failure cyclic load level to 10% of the applied bearing pressure.

The earthquake acceleration coefficient for Perth is 0.09, and for Fremantle, WA slightly lower. A site factor of 1.5 (as defined in AS1170.4-1993) would also be appropriate which would give a design earthquake base shear cyclic loading of the order of 10 to 15% of the foundation design bearing pressure. This exceeds the cyclic load levels observed to cause liquefaction of saturated, uncemented calcareous silt under model foundations during centrifuge tests conducted by Finnie and Randolph (3). However, liquefaction of the model shallow circular foundations tested occurred after up to 200 cycles at 48 seconds per cycle which is a far greater duration of load than that of a seismic event.

For the Fremantle Iceworks site, the raft foundation reduced design bearing pressures on the soil stratigraphy significantly below those which would have been applied if pad and strip footings had been used, as assumed by the investigating geotechnical firm in their liquefaction assessment. The testing done by Finnie and Randolph (3), and the existing correlations relating cone tip resistance and liquefaction failure cyclic stress ratio do not account for any cementation of the soil. The upper, stiffer soil strata inferred to be 3.0m deep may effectively shield the weaker underlying material from load. If load spread is assumed, through the stiff soil layer, then the raft foundation load at the interface of the uppermost 'weak' layer identified represents an increase in overburden pressure in the order of 45%. This does not correspond to a large increase in load.

A valuable indicator of liquefaction potential is the measure of pore pressure dissipation. This can give a good indication of relative permeability, and volume change characteristics of the soil, which are very important for evaluation of liquefaction resistance. Pore pressure measurement can indicate whether the soil is free draining with a high permeability, not free draining with a low permeability, or a medium between the two extremes. This greatly influences liquefaction potential. However, no established correlations currently exist between the measured CPTU pore pressure response and liquefaction resistance. As the cone penetrometer used at the Iceworks Stage 3 site was not equipped with a piezometer, such data was not available for evaluation.

The 'low' loads applied to the soil strata by the Iceworks Stage 3 raft foundation, the lack of local correlations for CPT test results, and the absence of pore pressure dissipation information, make liquefaction potential of the soil stratigraphy very difficult to assess. The high cost of adopting piling, as a conservative response to concern of the potential for liquefaction, compared to the costs of a more detailed site investigation, clearly highlight the potential benefit of supplementary testing in addition to the three CPT tests conducted.

6 CONCLUSION

The Fremantle Iceworks project indicates a clear need for :

• Detailed geotechnical site investigative techniques, to determine foundation design parameters to a greater degree of accuracy where calcareous soils are anticipated. This is particularly true where ground improvement techniques are not appropriate, or where the insitu undisturbed soil is likely to have greater strength and stiffness than that when treated.

• The use of several different techniques to firstly provide confidence in foundation design parameters inferred from test data, and also to discourage adoption of grossly conservative design parameters due to lack of information.

• Emphasis to Clients and Developers of possible enormous project cost savings if foundation design parameters accounting for cementation can be determined within an acceptable level of accuracy. The need for additional testing to determine the presence of soil cementation, and analysis to accurately model such increase in soil stiffness for foundation design needs to be stressed.

• The development of an accessible database of local experience, and correlations for onshore calcareous sites, in order that this experience can benefit the engineering of appropriate foundation systems, and ultimately the greater community.

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8 LIST OF FIGURES