LIQUEFACTION POTENTIAL ASSESSMENT AND PILE FOUNDATION DESIGN FOR HIGH EFFICIENCY GAS TURBINES AND COMPRESSORS IN PERTH

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ABSTRACT

This paper presents a case study of liquefaction potential assessment and pile foundation design for two high efficiency gas turbines and two gas compressors in Kwinana, Perth. Firstly the complexity of the local geology is described with a detailed discussion on the geotechnical risks of the potentially liquefiable loose sands or silty sands of 2 to 3m thickness at varying depths overlying the Tamala Limestone Formation which has potential cavities. The design criteria and the adopted geotechnical parameters for the developed geotechnical model are then discussed. A liquefaction potential assessment approach has been proposed after critical review of the currently published, with an emphasis on the importance of evaluating a suitable earthquake magnitude for a project site. Based on the results of liquefaction potential evaluation and the risk assessment of a number of options, pile foundations have been adopted for the gas turbine generators. A piling strategy has been developed for analysis, design, installation and testing for the proposed Franki piles founded on the Tamala Limestone Formation. The gas compressor raft foundations are to be founded on piles at a shallower depth above the liquefiable loose sandy layer to mitigate the potential risks of loosening/degradation effect of the cemented sands induced by the dynamic loads during the compressor operation. The assessed differential settlement induced by the potentially liquefiable sandy layer has been taken into account by the structural engineer for his detailed design and articulation of the reinforcement.

1 INTRODUCTION

Liquefaction potential assessment has become increasingly important with urban development within the areas of high geotechnical constraint around the world. Although a large number of researchers and engineers have attempted to establish the best way to evaluate the liquefaction potential of a particular site, it is often found that the input parameters required for the assessment are not readily available. The latest edition of Australia Standard AS1170.4 provides some guidance to the seismic design of critical infrastructure, which must meet a 1 in 2000 year return interval in evaluating the maximum ground acceleration. However it lacks a detailed method to determine a suitable magnitude of design earthquake (M) for the liquefaction potential assessment of a project site.

Machine foundation design requires an understanding of the interaction between soil and structure and the behaviour of soil/rock under the dynamic loads of varying characteristics (Prakash, 1981; Sydney University, 1983). For engineers it is of great challenge to use a simplified model to evaluate the response of the machine foundations due to lack of input parameters or budget constraints. Although true three dimensional numerical modelling has been getting more and more popular it is often too time consuming to use for some projects or not practical owing to a lack of reliable input parameters.

This paper presents the Kwinana High Energy Gas Turbine Project as a case study for discussion of the above two issues and an expansion on the work undertaken as part of the project. It presents the methodology used to design for static and cyclic loading, a discussion of design earthquake magnitudes, and the approach used to assess liquefaction potential and its impacts on the design. Finally, the pile design strategy is discussed so as to reduce geotechnical risks for the foundation system.

2 PROJECT DESCRIPTION

The Kwinana High Efficiency Gas Turbine Project is a power generation asset replacement project for Verve Energy in Perth. The overall scope of the project comprises the replacement of existing generation assets with two new gas turbines at the current Stage B boiler location within the Kwinana Power Station. The project also involves ancillary plant works including gas compressors, sea water cooling plant and associated electrical and instrumentation equipment. A locality map and the existing power plant layout plan are shown in Figure 1.

Hyder Consulting Pty Ltd was engaged by United Group Limited (UGL) to provide geotechnical input to the detail design of foundation systems for the Gas Turbine Generator (GTG) and Gas Compressors (GC) on the site. The foundation of the Gas Turbine Generator and other associated structure comprised a piled foundation formed by a reinforced concrete slab fifty eight (58) piles whilst the foundation of each of the two Gas Compressors consisted of a reinforced concrete foundation with six (6) piles.

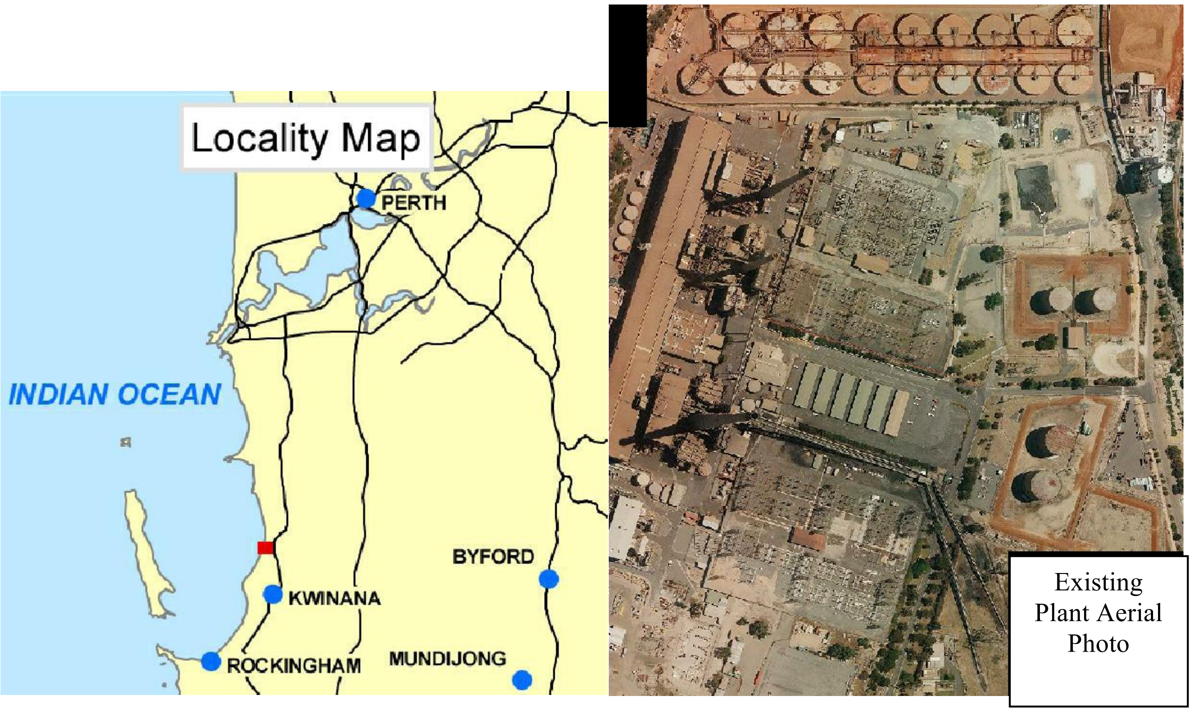

Figure 1: Locality map and existing power plant layout plan (Source: GHD, 2009).

3 LOCAL GEOLOGY AND SITE CONDITIONS

The regional geology in the area of Kwinana, Perth comprises the Quindalup Dune System, which consists of Safety Bay Sand and possibly the Becher Sand Unit underlain by the Tamala Limestone. The Tamala Limestone grades into the Ascot Formation underlain by the Osborne Formation.

Geotechnical site investigations available showed that the conditions encountered on the site matched the local regional geology. There is a layer of pavement and fill of 1 m to 2 m constructed above the original ground surface. The Safety Bay Sand unit varied in depth from 1.7 m to 4.5 m, changing to the Becher Sand unit which extends to a depth of about 17 m. The Tamala Limestone Formation with an overall thickness in the order of 9 m was intersected at depths between 16.2m and 17.5 m, usually contains a varying degree of cavities and is underlain by the stiff to hard clay of the Osborne Formation. It was noted from the investigation data that the Becher Sand unit at varying depth of about 12 m to 17 m became very loose to loose, with the standard SPT N value of 1 to 7, to the top of the Tamala Formation.

The groundwater table was encountered at the test locations and it varies around 3 m depth from the surface. This is in line with the mean sea level given the close proximity of the project site.

A summary of the geotechnical profile at the project site is presented in Table 1.

Table 1: Geotechnical profile on the Kwinana site

Geological Unit	Depth Range (m)	Description
Unit 1	0-2	Fill: Sand and Gravelly Sand, dense to very dense
Unit 2	0-12	Safety Bay Sand: Sand grading Silt, medium dense to dense
Unit 3	12-17	Becher Sand: Silty Sand, very loose to loose
Unit 4	17-28	Tamala Limestone: Low to high strength Limestone, contains cavities
Unit 5	28-40	Osborne Formation: Clayey Silt and Sandy Clayey Silt, very fine grained, medium plasticity, very stiff to hard

4 GEOTECHNICAL UNITS AND DESIGN PARAMETERS

Following our review of the geotechnical conditions on the site and the available geotechnical testing results presented in the relevant reports we have rationalised a geotechnical profile with specific Units and the associated geotechnical design parameters. A summary of the thickness of each Unit and the relevant geotechnical parameter adopted for the design is presented in Table 2. These parameters are generally in line with the recommended values presented by GHD (2009), with some parameters derived using published data or information from previous projects. Note that both upper and lower bound parameters of the medium dense to dense sands and limestone were proposed for design so a sensitivity analysis could be undertaken to account for the potential variations in these parameters across the project site.

Table 2: Adopted geotechnical parameters for each Unit

Geological Unit	Unit Thickness (m)	E_v (1) (MPa) Lower /Upper	E_h (2) (MPa) Lower /Upper	ν (3)	Φ' (4) (Deg)	$Φ_{int}$' (5) (Deg)	Su (6) (kPa)	γ (7) (kN/m^3)	k_s (8)
Unit 2	11.4	20/50	16/40	0.35	38	26		19	0.4
Unit 3	4.6	5	4	0.3	30	22.5		16	0.5
Unit 4	12.6	100/300	80/240	0.3	45	30		20	0.5
Unit 5	11.2	50+6.25/m	40+5/m	0.5			200+25/m	20	

(1) – Elastic modulus (vertical) (2) – Elastic modulus (horizontal) (3) – Poission's ratio (4) – effective friction angle
(5) – effective soil/pile interface friction angle (6) – undrained shear strength (7) – Unit weight (8) – lateral earth pressure coefficient

5 DESIGN CRITERIA

The proposed foundation systems for the gas turbine generator and gas compressors are expected to experience both static and dynamic loads during their operation.

The static deflection limits were specified by the client as a maximum of 25 mm over 25 years and a 1/300 maximum gradient due to differential settlement.

The dynamic loads resulting from the gas turbine generator usually have a number of motions (Sydney University, 1983) and each of the motions has to be considered in its foundation design. Some of the specific limits provided by the supplier of the gas turbine generator and gas compressor are as follows:

- Double amplitude of the Gas Compressor foundation plane vibration must be less than 10 micron
- Natural frequency of the Gas Compressor foundation to be 30% more or less than the normal rotor speed.
- The peak vibration velocity at any point of the Gas Turbine Generator foundation should be less than 1.5 mm/sec at the rated turbine speed of 3600 rpm.

In addition, the proposed Kwinana HEGT Project will be part of Perth peak power supply network and the relevant structural elements and their foundations are considered to be "Critical Infrastructure". Therefore the foundation and superstructure must be designed to account for the potential impacts resulting from the earthquake loading.

6 DESIGN CONCEPT DEVELOPMENT

At the tender stage a number of options were considered for the foundation system for both gas turbine generators and gas compressors. These included: 1) shallow raft foundation, 2) steel driven pile foundation and 3) bored pile foundation. Due to the presence of existing driven steel H-pile for the current power generation facilities, the liquefaction potential of loose sands above the limestone and the complication of potential cavities within the limestone formation it was decided to use enlarged base Franki piles. This foundation concept is similar to that used for foundations on an adjacent site. It was noted that the concept proposed by the preliminary geotechnical report recommended a piled foundation should be terminated well above the very loose to loose sand layer so that the high stress at the pile tip will not result in relatively high settlement. This recommendation was based on the key assumption that the very loose to loose sand layer is unlikely to be liquefiable.

During the detail design phase we raised the liquefaction issue with the client, in particular with regards to the magnitude of the earthquake adopted for the liquefaction assessment and its return period for earthquake. It was confirmed by the client's engineer that the liquefaction of the loose sand layer is likely when a return period of 1 in 2000 years was considered. Based on the potential for liquefaction, and the assessment of the site conditions the Franki pile option was adopted for the GTG and GC foundations. The GTG piles will be founded on the limestone and the GC piles will be founded at a shallow depth, above the loose sand layer.

The overall layout plan for the large GTG foundation is shown in Figure 2. A simplified raft model for the two main GTG, as shown on Figure 2, was adopted to allow for analysis.

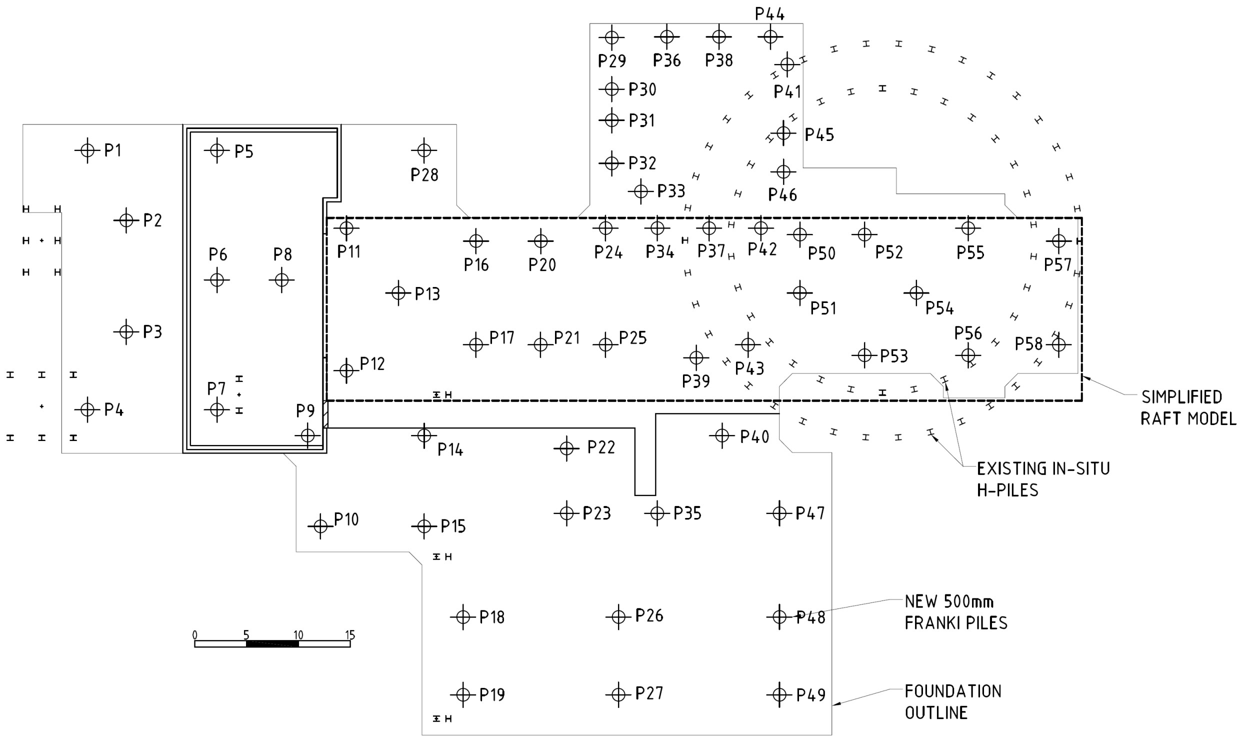

Figure 2: Gas Turbine Generator foundation pile layout.

7 FOUNDATION DETAIL DESIGN

7.1 ANALYSIS METHODOLOGY

Due to complexity of the dynamic motion of the piled foundations for GTG and GC, the program Strand was selected for both static and dynamic analyses. Given the time and budget constraints a simplified Strand model was established for the detail design rather than a full three dimensional finite element model. However this simplified model required the input parameters of the piles as "springs". These spring values were calculated using commercially-available program REPUTE as the geotechnical input. An iterative approach was adopted in order to come up with a robust design. The primary procedures involved in the modelling may be summarised as below:

1. The single pile stiffness under both vertical and lateral loading was initially calculated by a simplified elastic method using the lower and upper bound parameters for the adopted geotechnical profile.
2. The calculated pile stiffness values were then fed into the 3 dimensional Strand model to simulate all static and dynamic load cases.
3. The output was then used for further assessment of single pile using REPUTE for a single pile response to confirm the simplified method is reasonable.
4. The optimum pile layout was determined based on results of the Strand modelling and *in situ* constraints such as existing piles from the recently demolished boilers.
5. The simplified pile layout was then modelled in REPUTE as a pile group, to determine the group stiffness response to loading.
6. This stiffness was compared with the total group response in the Strand model to confirm the design.

7.2 PILE STATIC ANALYSIS

The geotechnical design analysis was undertaken using the program REPUTE to assess the response of a single pile and a pile group. REPUTE is similar to program DEFPIG and both are based on boundary element method in calculating the pile and soil interactions. REPUTE was chosen for this application because it is capable of handling 3-dimenional loading while DEFPIG can only compute loading in one plane. This feature of REPUTE allowed us to reduce the need for superposition to analyse a 3 dimensional problem.

One limitation of REPUTE is that analysis assumes a fully rigid pile cap. The pile cap for the GTG is 2000mm thick, and the smaller Gas Compressor slab is 1250mm thick, predominantly to increase the mass and reduce the impact of dynamic loading from the plant. The resulting stiffness leads to the rigid action of the pile cap.

The loads from Strand for the governing cases are presented in Table 3. Note that all the piles are modelled with a Young's modulus of 30,000 MPa.

Table 3: Pile diameter, length and loads adopted for REPUTE analysis

Loading Case	Pile Length (m)	Pile Diameter (mm)	Max Individual Pile Loads (kN)	Group Loads (kN)
GTG Vertical	16	500	1000	11529
GTG Horizontal	16	500	40	420
Gas Compressor Vertical	11	500	400	2270
Gas Compressor Horizontal	11	500	60	80

Figure 3 shows the predicted axial load distribution along the depth of a single pile under static loading. It is evident from this that the single pile is behaving as an "end-bearing" pile in that about 80% of the vertical load is transferred down to the Tamala Limestone at the bottom of the pile. This also highlights the importance of testing the installed piles to ensure the bearing stratum below the base of the pile has sufficient capacity. It is critical to prove on site there will be "no cavities" within the zone of influence below the pile base which could compromise the pile capacity.

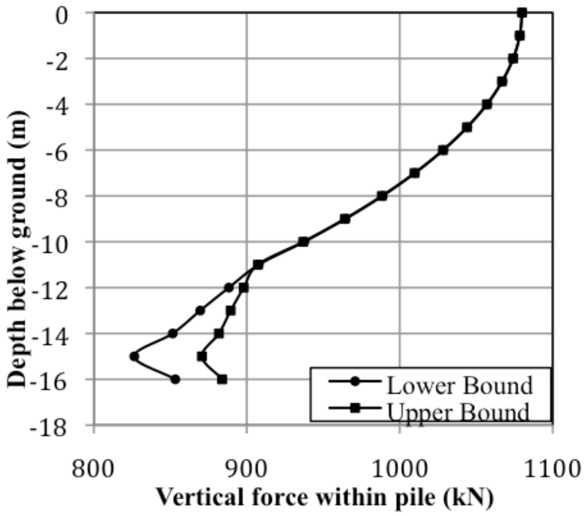

Figure 3: Single pile axial load distribution with depth

Figures 4a and 4b present a comparison of the single pile stiffness with the pile group stiffness for the Gas Turbine Generator. Based on the analysis discussed above, the vertical displacement of a single pile is expected to be in the range of 5 mm to 10 mm under the maximum single pile load of 1000 kN.

In comparison, the calculated vertical displacements of the pile group under the total pile group load of 11529kN is predicted to be about 3 mm to 8 mm for the upper and lower bound stiffness parameters set out above.

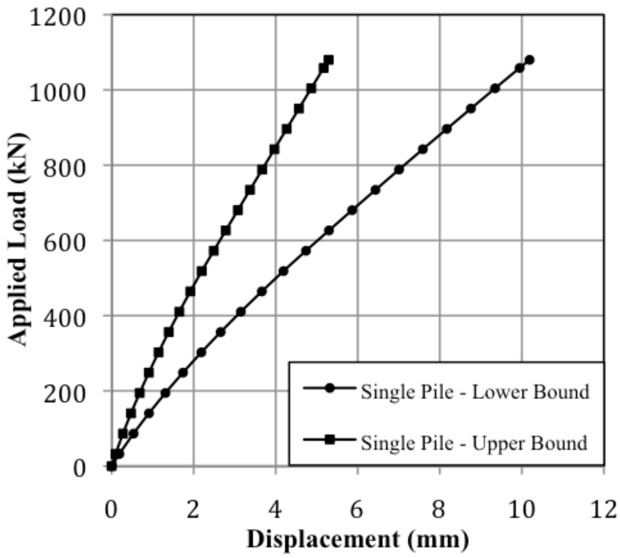

Figure 4a: Load-displacement response of a single pile

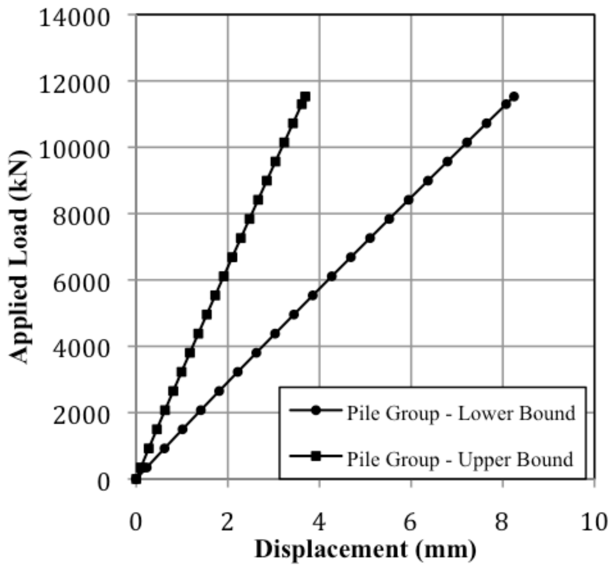

Figure 4b: Load-displacement response of pile group

Based on the load-displacement response shown in Figures 4a and 4b above, the spring stiffness was calculated for single and pile group responses as set out in Table 4. The single pile spring values were input into UGL's 3-dimensional model used for analyses. The pile group response predicted by REPUTE was then compared with the group response under the total loading in the 3-Dimensional structural model. This process provides an iterative check to confirm that the soil/structure interaction is modelled in a suitable manner.

Table 4: Spring values used for structural design

GTG Vertical Spring of a Single Pile (Static)	100,000 kN/m to 200,000 kN/m
GTG Lateral Spring of a Single Pile (Static)	10,000 kN/m to 15,000 kN/m
GTG Vertical Spring of a Pile Group (Static)	1,300,000 kN/m to 3,000,000 kN/m
GTG Lateral Spring of a Pile Group (Static)	220,000 kN/m to 500,000 kN/m

7.3 DYNAMIC PILE DESIGN

The pile stiffness under dynamic loads will depend upon the nature of the loading and ground conditions. Often it is not easy to conceptualise a model to simulate all the load cases in a simplified "spring" model. However for the prevailing load combinations it is reasonable to look into the correlation between the static modulus and dynamic modulus for each stratum supporting the piles. Based on downhole seismic surveys completed in two (2) boreholes on the Kwinana HEGT Project, the ratios between static and dynamic modulus for the primary geological units are presented in the Table 5. Note that the upper bound static Young's modulus of 75 MPa for Unit 2 was only recorded a couple of metres below the interface between Unit 1and Unit 2.

In reference to Fang, 1991 it is reasonable to adopt dynamic spring values which are of the order of 5 to 10 times that under static load. Dynamic group stiffness was assessed using REPUTE, with dynamic modulus equal to 10 times the static modulus previously used in analysis. A comparison of the static and dynamic results for the GTG pile group is presented in Figure 5.

Table 5: Comparison of static and dynamic modulus

Geological Unit	Static Modulus (MPa)	Dynamic Modulus (MPa)	Ratio of Dynamic Modulus to Static One
Unit 1	60-75	250-1000	4.2-13.3
Unit 2	15-75	470-780	23-52
Unit 3	3-7.5	440-570	76-147
Unit 4	120-320	1,300-3,100	6.9-17.2

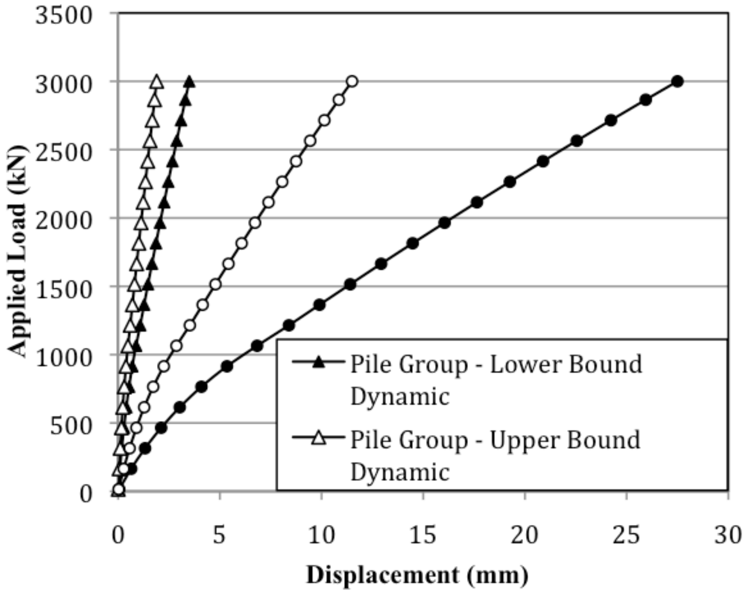

Figure 5: Comparison of static and dynamic stiffness

The upper and lower bound spring stiffnesses based on the analysis shown in Figure 5, are presented in Table 6. It can be noted that the calculated vertical spring values of the pile group under dynamic loading is about 3 to 5 times that under static loading, while the lateral spring values under dynamic loading are approximately 5 times that under static loading. This is in line with our experience on previous projects.

Table 6: GTG calculated dynamic group stiffness

GTG Vertical Spring of a Pile Group (Dynamic)	6,000,000 kN/m to 9,000,000 kN/m
GTG Lateral Spring of a Pile Group (Dynamic)	1,300,000 kN/m to 2,500,000 kN/m

8 ASSESSMENT OF LIQUEFACTION POTENTIAL

8.1 ASSESSMENT METHODLOGY

As discussed in the previous sections liquefaction is one of the critical issues considered in the GTG and GC foundation design. An assessment of liquefaction potential has been carried out at the Kwinana site based on the available geotechnical investigation information. The assessment has been carried out using the CPT and SPT methods proposed by T.L.*Youd et al.*, 2001. The fundamental basis of these methods lies in the work of Seed and Idriss who presented their “simplified procedure” in Seed and Idriss, 1971. In this approach, the cyclic stress ratio (CSR) is compared with the cyclic resistance ratio (CRR) at a point of interest to determine a factor of safety against liquefaction. The method has been refined and developed in a number of papers by Seed, 1979, Seed and Idriss, 1982 and Seed *et al.*, 1985. Further developments to this method were also made by Suzuki *et al.*, 1997 and Robertson and Wride, 1997. The more recent paper by Youd *et al*., 2001 presents a concise summary of the approach to determining liquefaction potential, and collates research in the area from various sources in one location.

The initial calculation of Factor of Safety (FoS) is undertaken for an earthquake Magnitude (M) of 7.5 as a benchmark, and magnitude scaling factors are used to adjust this result back to the design earthquake. In our liquefaction assessment we adopted the revised Idriss scaling factors, which are considered to be a lower bound for M<7.5 as proposed by Youd, 2001. It has been found that the liquefaction potential is highly sensitive to the specified magnitude of earthquake.

Despite this, there is no indication in AS1170 as to appropriate design magnitudes for earthquakes across Australia. A number of researchers have tried to establish the generalised relationship between the magnitude of earthquake (M) and the probability of exceedance. The paper entitled “Recurrence Relationships for Australian Earthquake”, by C.Sinadinovski and K.F.McCue, 2001, provided a useful correlation for Australian Earthquakes. Other studies by both Love, 1996 and Gibson, 1996, as presented by Mitchell & Moore, 2007, suggest similar relationships between the magnitude of earthquake and the probability of exceedance or return period for Adelaide. Based on these two papers and our recent study for the Port Botany Expansion project the magnitude of earthquake for the Kwinana site is assessed to range from 5.8 to 6.8 Richter scale with a probability of exceedance of 2% chance in 100 years or a return period of 1 in 2000 years. It should be emphasised that there is a great deal of uncertainty as to the appropriate curves for the project site and region due to the lack of the measured earthquake data at the higher end of the earthquake magnitude scale.

Other input required for the liquefaction assessment includes the peak ground acceleration coefficient of 0.09 and a site factor of 1.8 for a return period of 1 in 2000 years.

8.2 RESULTS OF LIQUEFACTION ASSESSMENT

Figures 6 and 7 show the calculated FoS against liquefaction at various depths at all four borehole locations using the Seed SPT and CPT methods for earthquake magnitudes of 5.8, 6.3 and 6.8. It can be seen that, depending on the method, there is a potentially liquefiable zone at varying depths from 11m to 16m, with a calculated FoS less than unity for magnitudes equal to or greater than 6.3. This means that for earthquake magnitudes of 6.3 and 6.8 there is potential for liquefaction of the sandy material between depths of 11m to 16m. When the earthquake magnitude is 5.8 only one of the boreholes shows a potential for liquefaction when using the SPT method, and none for the CPT method.

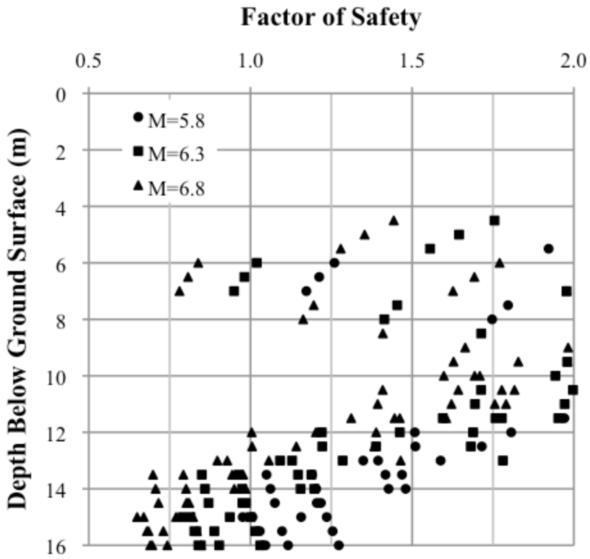

Figure 6: Calculated FoS using the Seed SPT method

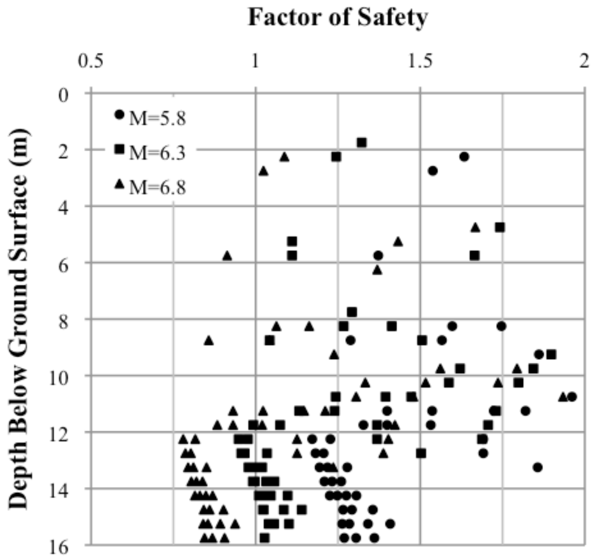

Figure 7: Calculated FoS using the Seed CPT method

Iwasaki *et al.*, 1984 proposed an approach based on the SPT-N and mean particle diameters of the soil which was correlated against data across 87 sites during six past earthquakes. This method is more conservative than those proposed by Youd, as can be seen in Figure 8 where Iwasaki's Liquefaction Resistance Factor (F_L) is plotted alongside the Factor of Safety based on the CPT and SPT methods proposed by Seed.

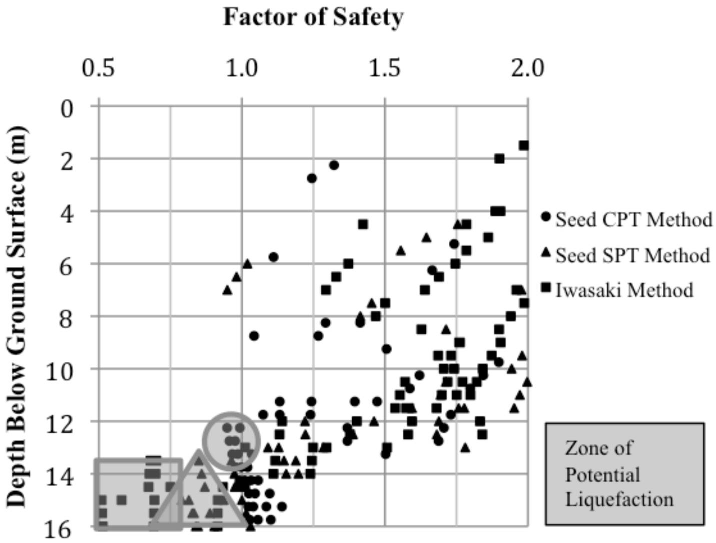

Figure 8: Iwasaki method compared with Seed methods for M=6.3

It is evident from Figure 8 that the zone and degree of liquefaction varies with the method used in the assessment. The zone of liquefaction for each method is represented by a shaded area matching the legend. The two methods based on SPT-N values show a reduced FoS to that based on CPT data. This is consistent with the findings of Mitchell & Moore, 2007, who reported FoS based on SPT-N to be on average 67% less than that of CPT's in loose silty sands under the water table. It was concluded from their investigation of a number of sites, that Seed's CPT based method is more suitable than the similar SPT-N based method in these materials.

While the SPT-N based F_L value results in significantly lower values to the other calculated Factors of Safety, the benefit of Iwasaki's approach is the inclusion of the Liquefaction Potential Index, which provides an estimation of the severity of the earthquake on structures at a given site.

This method gives a weighted significance to zones of liquefaction, whereby the impact is reduced with depth from the surface. This reduction of impact based on depth considers the fact that liquefaction induced settlements at increasing depth will have a reduced impact on surface structures. The calculated Liquefaction Potential Index at the four boreholes completed on site ranges between 1.5 and 4.5, which corresponds to a low risk of damage to surface structures.

Based on the results of the above assessments we consider that there is potential for liquefaction of a layer of the sandy material at depths of 11 m to 16 m, however it poses a relatively low risk to structures at the surface. Given the nature of critical infrastructure, it is still considered prudent to take this possible liquefaction impact into account in the design of the GTG foundation system.

The GTG foundation is a large stiff element, and it was considered that possible differential settlements due to liquefaction could not be efficiently managed across such a large structure if piles were founded at shallow depths.

8.3 GAS COMPRESSOR LIQUEFACTION SETTLEMENT ASSESSMENT

The design of the much smaller gas compressor foundation system was carried out in the same manner as discussed above for the Gas Turbine Generator. One exception is that due to the much smaller vertical loads it was considered possible to found the piles 3m above the loose sand layer. Founding the piles above the potentially liquefiable layer required us to assess the possible settlement of this layer under liquefaction conditions.

Unfortunately, the CPT in close proximity to the GC does not reach sufficient depth to disprove the presence of the loose sand layer seen at other site investigation locations. Should the magnitude of earthquake discussed above occur, it is likely that this layer will liquefy and undergo subsequent compaction.

The settlement of wet soils induced by liquefaction has been discussed by Tokimatsu & Seed, 1987 and Ishihara and Yoshimine, 1990 and the US Army Corps of Engineers, 1994. The effects of this compaction were assessed based on the method presented by the US Army Corps of Engineers, 1994.

The liquefiable layer based on CPT results, varies in thickness across the site from 2 m to 3 m. Based on the US Army Corps method, an absolute total settlement of 100 mm to 150 mm is expected as a result of liquefaction. This is consistent with the typical 5% settlement proposed by Ricardo, 1994. To determine differential settlements, we considered the distance between these thicknesses. The sand layer on site is relatively uniform, and the variance from 2 m to 3 m thickness occurs over a distance of almost 40 m. Considering the relatively low risk of earthquake induced damage to the surface structures based on Iwasaki's method, and the relative short length of 10 m for the Gas Compressor foundation we scaled the differential settlement down to 10 mm to 20 mm by linear interpolation. This level of differential settlement falls within the limits of the foundations capacity.

9 PILING STRATEGY ADOPTED

Based on the analysis and assessment described above three key factors were considered in the pile foundation design:

- The impact of the potential liquefiable sandy soil on the top of the limestone on the pile capacity.
- The potential settlement when the loose sandy soil is liquefied.
- The presence of the potential cavities within the limestone formation.

To cater for the above three key constraints the following mitigation measures have been employed for the GTG foundation:

- To install the piles through the very loose sand layer and to found in the limestone.

- To confirm the pile capacity by appropriate level of testing on site to ensure each pile has achieved its capacity.
- To evaluate the impact of down drag during and post the earthquake on the pile foundation.
- To provide a robust structural connection between pile head and slab in case any of the piles in a group have a lower capacity

For the GC foundation the following were considered:

- Piles are to be used to cater for the potential degradation of cemented sands under dynamic loading.
- Piles are to be installed at least 2 m or more above the liquefiable sands so that sufficient pile capacity is achieved under static loading.
- To take account of the assessed differential settlement into the structural design of the piles and slab

Based on all the factors described above the Gas Turbine Generator design includes fifty eight Franki piles with 500 mm shaft diameter and minimum base of 600 mm equivalent diameter, founded at the top of the Tamala Limestone formation. A proportion of piles were PDA tested on site to confirm the required pile capacity which will be undertaken by a specialist contractor. The results are then used to validate the base bulb driving process and capacity evaluation.

The slab for the GTG was designed as a highly redundant structure with 58 piles. The slab was designed with the ability to shed load to adjacent piles for the case where a pile has much less capacity or undergoes relatively larger settlement. Sufficient reinforcement was introduced at the pile and slab connection to deal with such load transfer. This design strategy has made it possible to reduce the testing frequency, and minimise the risk of inadequate overall pile foundation capacity when founded on limestone with potential cavities.

At the Gas Compressor location, the vertical loads are significantly lower than for the GTG. As such, the strategy was to construct a slab with six number of 500mm diameter Franki piles founded 2-3 m (4-5 times the pile diameter) above the soft sand layer at a depth of approximately 11 m. In addition to the static pile design, a liquefaction assessment was carried out and a potential differential settlement of about 20 mm under liquefaction conditions was considered in the structural design of piles and slabs.

10 CONCLUSIONS

Based on the case study presented in this paper it is recommend that liquefaction potential of silty sands be assessed using currently available methods based on CPT profiles rather than those using SPT blow counts. If both sets of data are available, a comparative study is considered to be the prudent approach. Particular attention should be paid to the determination of suitable earthquake magnitude in that the liquefaction potential assessment is highly sensitive to this parameter.

A full three dimensional dynamic model of the foundation structure, soil and rock is considered desirable, should all the input parameters and the budget be readily available. However, dynamic loads on a machine foundation can be considered by engineers using the simplified method, provided appropriate "spring" values are determined. This requires an analysis approach that considers geotechnical and structural interaction and both engineers should work closely so that the best solution can be achieved.

When the piled foundation is to be terminated above potentially liquefiable soils the total and differential settlements of the subject foundation should be carefully assessed and these movements taken into account in the structural design and detailing.

At the time of writing this paper the piles have been installed and tested to be successful. It is found that the proposed procedures and method adopted in this project could be of use for similar foundation design.

11 ACKNOWLEDGEMENTS

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