Foundation design and performance of 8,300 tonne storage tanks in Kwinana, WA

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ABSTRACT: This paper presents design considerations of the foundation system for three 8,300 tonne storage tanks founded on calcareous deposits in Kwinana, Western Australia. Due to stringent settlement criteria, shallow foundations would not be suitable, and a CFA pile option was considered to be the most economical solution due to its common availability in the local WA market. High strain dynamic pile tests demonstrated that the total capacity of the test piles was higher than the design ultimate geotechnical strength. The mobilised end bearing is higher, but the shaft friction is lower than the design values. Although predicted overall settlements were within the design limits, settlements observed during the hydrotests were larger than the predicted values. The observed settlements are interpreted as resulting from compression of varying grades of the Calcarenite in concert with interbedded sand layers within the depth of influence zone beneath the large diameter tanks.

1 INTRODUCTION

Calcareous deposits are known to be non-uniform and highly variable depending on their origin, age, mineral composition and grain size. The Kwinana Indus-trial Area (KIA) is one of Western Australia's (WA) most important strategic industrial zones, and is underlain almost entirely by Calcareous Sand over Ta-mala Limestone. Despite many major heavy structures and facilities having been built and operated in the KIA, limited case histories on their foundation de-sign and performance have been published.

An import facility for a chemical product, including storage tanks, hot oil slab, pump slab, gantry weigh bridge, utility areas and paved driveways, has been constructed at a site in Kwinana. This paper presents design considerations of the foundation system for three 8,300 tonne storage tanks founded on the calcareous deposits in Kwinana.

2 GROUND CONDITIONS

2.1 *Geology*

Published geological maps for the area depict the land as being underlain by Safety Bay Sand followed by Tamala Limestone. Safety Bay Sand is described as "*calcareous sand, white, medium grained, rounded quartz and shell debris, well sorted, of eolian origin*". The deeper geological formation, at nominally 27m below existing ground level, is reported to be Meso-zoic shales and siltstones.

2.2 *Field investigation*

Field investigation was carried out, which comprised 2 machine boreholes with Standard Penetration Tests (SPTs) conducted at 1.5m intervals, and up to 16 Cone Penetration Tests (CPTs). Samples were collected for engineering classification and point load tests were performed on the core samples to assess the quality of the Limestone.

The overlying Calcareous Sand (carbonate contents 70-80%) is generally fine to medium grained (average particle size, D_{50}, of 0.19mm with uniformity coefficient, C_u, of 2), angular to sub-rounded with traces of shell fragments. Transition between the Calcareous Sand and the underlying Limestone (classified as Calcarenite) is variable and this zone may contain more silts and gravels. The Calcarenite unit is highly variable in nature and could range from in-tact rock to uncemented material with total core loss or gravel as recovery.

2.3 *Groundwater*

The Perth Groundwater Atlas suggests groundwater levels varying from approximately RL0.5m AHD to RL2m AHD. This is consistent with the groundwater observed within the CPTs and boreholes at depths of approximately 2m to 3.5m below the existing ground surface levels during investigation.

2.4 *Subsurface profile*

A typical profile of CPT tip resistance is presented in Figure 1. It is observed that the cone tip resistance (q_c) generally varies between 5MPa and 20MPa. A denser crust layer is found near the ground surface and above the groundwater table. A looser layer is typically found at the transition above the Calcarenite and exhibits a slightly higher friction ratio.

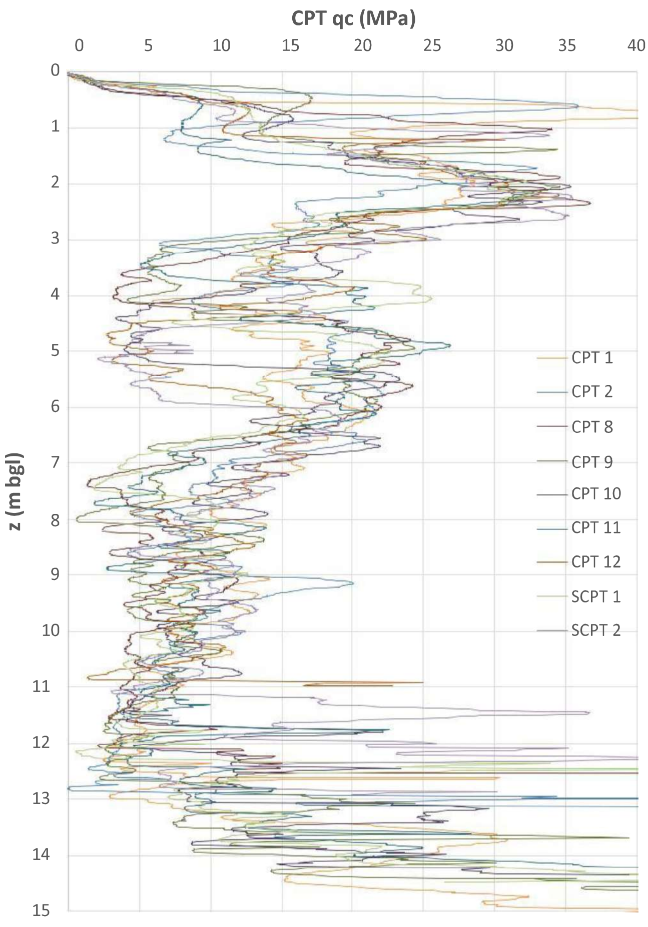

Figure 1. CPT profiles.

2.5 *Design parameters*

For the purposes of geotechnical design calculations, strength and stiffness parameters have been interpreted for each of the soil units, based on the results of the geotechnical investigation, published literature on similar soils and case histories. The adopted geotechnical design values of the in-situ materials are summarised in Table 1.

3 DESIGN CRITERIA

3.1 *Loading criteria*

The storage tanks are 22m in diameter and 20m in height. The loading conditions of the tanks, that form the basis of the design, are presented in Table 2.

Table 1. Geotechnical Design Parameters

Description	q_c (MPa)	’ (deg)	c’ (kPa)	E_{50} (MPa)	E_{ur} (MPa)
Calc Sand (VL to L)	<5	30	0	3-15	9-45
Calc Sand (MD)	5-15	34	0	15-45	45-135
Calc Sand (D to VD)	>15	37	0	45-100	135-300
Calcarenite	NA	40	5	1000	-

Notes:
q_c = cone tip resistance, ’ = effective angle of friction; c’ = effective cohesion; E_{50} = secant Young’s Modulus at 50% of failure stress; E_{ur} = unload/reload Young’s Modulus

Table 2. Loading Criteria

Stage	Working Load
1. Existing ground conditions	0kPa
2. Compaction completed	0kPa
3. Install tank foundation	0kPa
4. Base slab and tank preload	28kPa for 6 months
5. Hydrotesting	203kPa for 24 hours
6. Unload and connect piping*	28kPa
7. Fill with chemical product	235kPa for 2 months
8. Tanks in use	28kPa to 235kPa every 2 months 300 cycles over 50 years 600 cycles over 100 years

* Settlement limits applied from the completion of this stage

3.2 *Settlement criteria*

The storage tanks were designed to comply with the following criteria for the 25 years design life of the structures:

- Total vertical settlement shall not exceed 20mm following installation of the piping connections.
- Differential vertical settlement shall not exceed 20mm.
- Horizontal movement shall not exceed 10mm in any direction.

4 FOUNDATION OPTIONS

4.1 *Raft foundation on untreated ground*

A preliminary assessment of the long-term settlements for the storage tanks suggested that the tank edge settlements would exceed the allowable limit of 20mm. This is in line with the observations by Hill-man et al. (1999) on similar storage tanks supported on shallow foundation in the Kwinana area.

The initial prediction prompted the assessment of various deep foundation options, including driven precast concrete and steel pipe piles, continuous flight auger (CFA) piles, Controlled Modulus Columns (CMC) and Vibro Stone Columns (VSC), which are discussed below.

4.2 *Driven piles*

Driven concrete and steel piles are likely to have a lower shaft resistance (compared to bored piles) due to the presence of crushable calcareous sands and significant degradation of shaft resistance following driving (White & Bolton, 2004). In addition, corrosion of the steel piles and shallow refusal of the pre-cast concrete piles in a dense sand layer could be problematical.

4.3 *Continuous Flight Auger (CFA) piles*

CFA piles would need to be carefully installed to pre-vent ‘flighting’ of the piles, where the sand surrounding the pile shaft is loosened by the action of over rotation of the auger during pile installation. High capacity rigs and a low design penetration depth into the underlying limestone were recommended to reduce the risk of ‘flighting’.

4.4 *Controlled Modulus Columns (CMC)*

CMC are unreinforced concrete piles which are typically installed using a CFA piling rig. The columns are not designed as conventional piles to transmit all the loads to the competent stratum but rather as rigid inclusions to improve the ground stiffness and reduce settlements. A compacted stone blanket, between the cut-off level of the CMC and the underside of the raft structure, is normally formed as a load transfer plat-form.

4.5 *Vibro Stone Columns (VSC)*

The formation of stone columns is carried out using a heavy vibrating poker to displace the in-situ ground and compact the imported stone. However, based on the CPT traces, the in-situ sand below the tanks may be too dense to be effectively displaced with a vibrating poker. Therefore, the use of stone columns was not considered to be practical for this development.

The CFA pile option was considered to be the most economical solution due to the common availability of the system in the local WA market.

5 PILE DESIGN

A representative ground model and pile design values for CFA piles are presented in Table 3. Note that the adopted design values of shaft resistance for the Calcareous Sand are about 30-40% lower than the values recommended by Bustamante and Gianeselli (1982) for the hollow auger bored piles (Category IA) in (siliceous) sand.

A geotechnical strength reduction factor, g, of 0.70 for large pile groups, has been applied on the basis that all piles will undergo sonic echo integrity testing and a minimum of 5 working piles will undergo high strain dynamic load testing. The design geotechnical strength, $R_{d,g}$, of the specified CFA pile is therefore estimated to be about 2,200kN.

Table 3. Pile Design Values

Description	Average Thickness (m)	Shaft Resistance (kPa)	End Bearing (kPa)
Calc Sand (VD)	1.0	80	-
Calc Sand (MD)	2.0	25	-
Calc Sand (VD)	2.0	90	-
Calc Sand (MD)	4.0	35	-
Calc Sand (VL-L)	2.0	20	-
Calc Sand (MD-VD)*	2.0	75	-
Calcarenite	0.5	120	7000

* Highly variable weathered Calcarenite

For each tank, a total of 50 CFA piles of 600mm diameter were designed to transmit the tank base loading to the underlying Limestone. Figure 2 shows a typical layout for the CFA pile option forming the tank foundation. The average pile length was estimated to be approximately 13m with a specified minimum socket length of 0.5m into the Limestone.

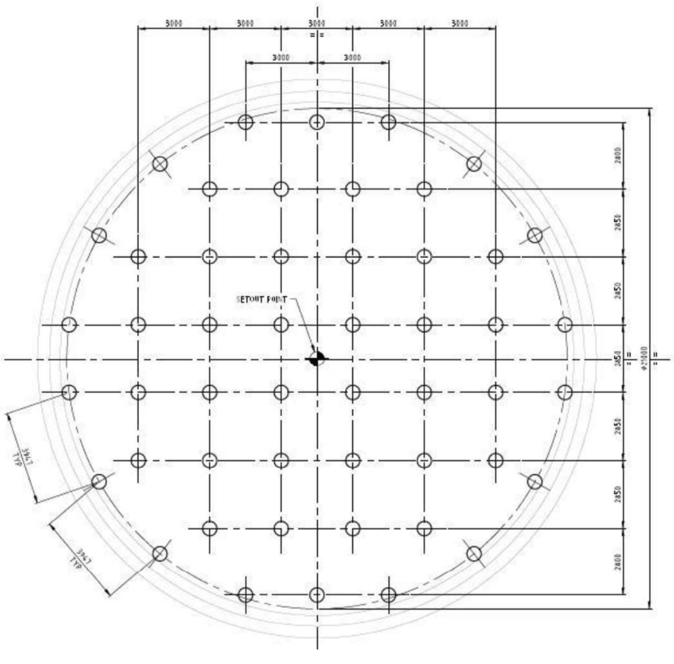

Figure 2. Piling layout plan.

6 PILE LOAD TESTS

6.1 *High strain dynamic pile tests*

High strain dynamic pile tests were performed on 5 selected CFA piles. The results were analysed using CAPWAP software and the output is summarised in Table 4.

Overall, the measured capacities are higher than the estimated design ultimate geotechnical strength, $R_{d,ug}$, of the piles. The CAPWAP results indicated that, while relatively high end bearing ($q_b \geq$ 7000kPa) was achieved, indicating that the piles were founded in the Calcarenite, the measured average shaft resistance was generally lower than the reduced shaft resistance adopted in the design

Table 4. CAPWAP results

Test Pile	Shaft Friction (kN)	End Bearing (kN)	Total Capacity (kN)	Settlement at Test Load (mm)
P23	1445	2406	3851	3.4
P69	1768	1968	3736	3.5
P120	1617	3002	4619	3.2
P133	1505	2372	3877	3.7
P157	1317	2911	4228	3.8

Back-analysis of the CAPWAP results suggests that the average shaft resistance for the Calcareous Sand in Kwinana area is about half of the values generally reported for siliceous sand. It is also important to note that the piles must penetrate through the layer of highly variable weathered Calcarenite and be founded in a more competent Calcarenite to achieve the end bearing resistance detailed above.

Although high strain dynamic pile tests are widely accepted in practice in Australia, static load tests are strongly recommended to gain a higher confidence on the pile performance.

6.2 *Sonic echo integrity tests*

Sonic echo integrity tests were performed on all working piles and the results indicated that no major defects were detected. This is an important QA/QC measure for the CFA piles when being constructed in highly variable Tamala Limestone.

7 HYDROTESTS

Following completion of the storage tank construction, hydrotests were performed to confirm the over-all integrity of the tanks. One critical observation made during the hydrotests was the settlement performance of the tank foundations. As described in Section 3, the foundations are required to provide adequate support to the tank under various loading conditions and to limit settlements to the specified values.

For each tank, 8 settlement monitoring points along the ring beam were established and settlements were measured. The tank edge settlements during the hydrotests were recorded as approximately 15mm for two of the tanks and approximately 10mm for the third tank. It was also recorded that the rebound during tank unloading was generally about 1-2mm and not more than 4mm at the most critical edge location.

This observed magnitude of settlement is larger than the predicted elastic shortening of the piles in combination with the estimated pile deformation based on the assumed stiffness of the Calcarenite. The observed settlements are interpreted as resulting from the compression of varying grades of the Calcarenite, in concert with interbedded sand layers, within the depth of influence zone beneath the relatively large diameter tanks.

The relatively low magnitude of rebound indicates that most of the compression incurred during the hydrotests is non-recoverable, and the subsequent re-loading is anticipated to be relatively stiff especially up to the hydrotesting pressure.

Observations during hydrotests do not include long-term secondary settlements under constant load and settlements arising from unloading-reloading of the tank during serviceability. However, it is anticipated that secondary settlements will not be critical based on the foundation behaviour observed from the hydrotests.

8 CONCLUSIONS

The following conclusions can be drawn from the design and testing of the tank foundations in Kwinana:

- The CFA piling system is commonly available in the local WA market and offers competitive pricing compared to other solutions.
- An automated monitoring system must be equipped on the CFA piling rig to identify and help prevent 'flighting' and other installation problems.
- The assumed pile design values must be verified through pile load tests particularly where variable ground conditions are expected.
- The average shaft resistance for the Calcareous Sand in Kwinana area is assessed to be about half that of the values generally reported for siliceous sands.
- High end bearing in the Calcarenite can only be achieved in a more competent stratum after penetrating through the highly weathered upper layer.
- The Calcareous Sand and the Calcarenite are relatively compressible during loading (as compared to siliceous sand) and the compression is largely non-recoverable upon unloading.
- The effects of pile group and the behaviour of soil/rock mass within the influence zone of the tank must be carefully assessed.

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