Sampling and Testing Cretaceous Claystone near Darwin, NT

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Summary: As part of a proposed $10 billion development of Timor Sea gas reserves, Woodside Energy Ltd in joint venture partnership with Shell Development Australia evaluated an onshore LNG processing plant to be situated at Glyde Point, on the Gunn Peninsula about 45 km north-east of Darwin, in the Northern Territory.

Gutteridge Haskins & Davey Pty Ltd was commissioned to perform a preliminary geotechnical investigation as part of the project feasibility study. The work included an assessment of the suitability of the site for the proposed development, to facilitate the determination of foundation design parameters for 120,000 cubic metre capacity LNG storage tanks and the prediction of settlements of shallow footings.

The site was found to be underlain by a monotonous sequence of weathered, lateritised kaolinitic claystone and shale, or clay shale, becoming montmorillonitic with depth. Difficulties experienced during the investigation included a slower than anticipated rate of drilling in "soft" rock, disturbance during drilling, remoulding of cored samples, and difficulty in recovering "undisturbed" samples.

The evaluation of the field and laboratory data included reconciliation of field and laboratory test results and charting variation in basic soil parameters with depth. A large discordance was found between various methods of strength and deformation characteristic evaluation. Due to lateritisation of an overconsolidated profile, the usual strength and depth relationships were not observed, and these anomalies had to be taken into account in developing a geotechnical site model.

The paper summarises the main conclusions drawn from the work, including identification of limitations of the investigation procedures and a recommendation of alternative testing methods for future detailed geotechnical investigations on the site.

1 PROJECT BACKGROUND

Details of the proposed Northern Australia Gas Venture (NAGV) were released for public access in 1999 by Woodside Energy Ltd and Shell Development Australia. The NAGV project was established in May 1997, to evaluate the feasibility of producing LNG from gas fields in the Timor Sea.

The feasibility study was finalised in December 1998, and included assessment of offshore production facilities, a 600 km subsea pipeline, and the proposed onshore LNG processing facility at Glyde Point.

The onshore plant site covers about 600 hectares, and was selected following consideration of land access in light of land claims, proximity to Darwin, marine access to deep water and environmental, and operational considerations regarding potential future development.

Process facilities were planned to comprise a LNG Plant consisting of two Propane-Mixed Refrigerant LNG trains, a 500 million standard cubic feet per day capacity domestic gas plant, and two train 5300 tonnes per day Condensate Stabilisation Plant. LNG storage facilities comprise two 120,000 cubic metre capacity containment tanks, with prestressed concrete walls and reinforced concrete base slab and roof, and condensate storage facilities comprising two 70,000 cubic metre floating roof tanks.

Relatively little existing data was available for the onshore plant site location, and substantial site survey and surface and subsurface investigations were therefore essential to the feasibility study.

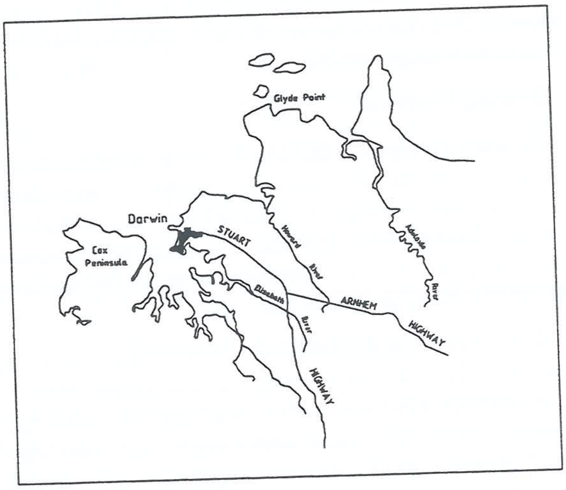

Figure 1: Locality Plan

2 GEOLOGICAL SETTING

According to published geological information (Pietsch, 1983 & 1985), the region is underlain by flat-lying Early Proterozoic and Cretaceous fluviatile, shallow-marine, and chemical sediments of the Bathurst Island Formation (BIF).

Glyde Point itself is underlain by a BIF member known as the Wangarlu Mudstone Member (WMM). The WMM is reported to be conformable with, lithologically similar to, and visually indistinguishable from an underlying unit called the Darwin Member (DM). Both units are reported to consist of weathered, kaolinitic claystone and minor, silty claystone, which becomes montmorillonitic at depth. The claystone is silicified in places and calcareous at depth, with shelly fragments commonly present. The lower boundary of the DM consists of a marker horizon of bioturbated material contained within thin carbonate beds. The combined thickness of the WMM and DM is inferred to be greater than 70 m.

3 FIELD INVESTIGATIONS

3.1 Locality Description

The Glyde Point site has low topographic relief, and is fringed on three sides by tidal mangrove swamps. Medium density vegetation comprising eucalyptus and mixed scrub plant species such as pandanus, melaleuca, grevillea and acacia is characteristic of the area. The coastal mangrove fringe is a saltwater crocodile habitat. Accordingly, the project-specific Health, Safety, Environment and Aboriginal Heritage Management Plan which was written to comply with client health and safety requirements also included a section titled "Crocodile Safety Plan", prepared in consultation with Wildlife Management International Pty Ltd and the Parks & Wildlife Commission of the Northern Territory. This document served as a training tool for the protection of field personnel.

3.2 Drilling Program

The site work comprised drilling 25 vertical boreholes on a grid pattern across the site, by washbore and wireline diamond drilling techniques. The drilling was performed by contractor, Bachy, who mobilised a TRH tractor-mounted rotary drilling rig, and the boreholes were drilled to varying depths up to a maximum of 60 m.

Each of the holes was drilled by washbore to a nominal depth of 5 m, then HQ diamond coring to the target depth. A polymer mud additive was used to stabilise the holes during washboring, however, water flush only was used during diamond drilling in weathered claystone, with the hole typically cased over the top 3 m. Sixteen boreholes were drilled to 10 m, five of which were drilled by washbore only to allow in situ testing.

3.3 In Situ Sampling & Testing

Recovery of "undisturbed" U65 tube samples was attempted in soil, however, the ground was too hard and sampling proved to be impracticable. Intact, "semi-undisturbed" samples for laboratory testing were recovered removing diamond drill core from the triple tube inner core barrel. After cleaning off the mud cake from the outside of the core, it was placed inside transparent lay-flat plastic, and taped and stored inside HQ-sized split PVC tubing for transport to the testing laboratory. Shrinkage cracking of the core tended to occur on exposure to air, due to the effect of sun-drying, therefore placement of moistened rags was necessary whilst the logging was performed, prior to sampling of the core.

Standard Penetration Tests (SPT) were carried out at 1 m intervals in each of the washbore holes, to depths of either 5 m or 10 m, depending on the depth of washboring. The testing was carried out in accordance with the Australian Standard procedure, using a drive hammer of 63.5 kg mass falling 760 mm with automatic trip mechanism. Disturbed SPT split-spoon samples were retained for laboratory classification testing.

Pocket penetrometer tests were carried out on the drill core.

4 SUBSURFACE CONDITIONS

4.1 Soil Profile

The borehole results revealed materials in the upper 5 m consisting of surficial colluvial lateritic gravels overlying inorganic, low to high plasticity, red-brown and orange mottled clays. The clays were typically very stiff to hard in consistency, in dry condition and characterised by variable weak iron-cementing.

Beyond about 5 m depth, the ground conditions comprised partly-indurated residual soil, generally red-brown, orange and grey mottled or grey, and often containing iron-oxide coated joints and microfractures in a random pattern and orientation. The indurated zones tended to be brittle and to fragment easily, and were assessed as being weak rock, with strength in the extremely low to low range. Induration can be attributed mainly to silicification, which has occurred through mobilisation of silica and precipitation through groundwater fluctuations within the soil profile (Nyland & Gerner, 1984; Clarke et al, 1999).

The nature of the materials is consistent with the Tertiary chemical weathering described by Pietsch (1983), and is typical of deep trizonal lateritised soil profiles found throughout the region, and comprises zones described on the basis of colour, soil mineralogy and formation processes as either ferruginous, mottled, or pallid. The uppermost layers are enriched in iron and aluminium oxides, and the underlying pallid zone material is depleted, as a result of leaching of iron and aluminium. Nyland & Gerner (1984) have previously reported the presence of rock within the profile. Lateritised siltstone is common in a hardened mottled zone, and silicified claystone, referred to locally as "porcellanite", and kaolinitised siltstone can occur within the pallid zone. The uppermost ferruginous zone also often contains an indurated, strongly-cemented layer of pisolitic laterite, of variable thickness and lateral extent, however, this layer was mostly absent from Glyde Point, and had probably eroded away, leaving behind the layer of colluvial gravel.

Significant loss of drilling circulation fluid occurred in one of the boreholes, the loss occurring within the mottled portion of the profile through sub-horizontal cavities up to 20 mm in diameter. Similar features, locally termed "rat holes", are reported to exist in the Darwin area and form active channels of high secondary permeability through otherwise relatively impervious materials (Nyland and Gerner, 1984).

The transition between the pallid zone and underlying bedrock was typically not sharply defined, and occurred between 20 - 50 m depth.

4.2 Bedrock

The bedrock was found to consist of dark grey interbedded claystone and shale. The claystone is massive, and based on simple field identification criteria ranges from extremely low to medium strength. The shaly zones characteristically had a penetrative horizontal fabric, and as a result of marked strength anisotropy drilling breaks were closely-spaced. The widely-used classification scheme proposed by Moye (1955) was used to describe degree of weathering, according to which the material appeared to be mainly in the extremely weathered to slightly weathered range. Since claystone and shale are composed of clay minerals, however, weathering is predominantly a process of reverse induration, and a clear distinction between the various grades of weathering is therefore difficult to make. An alternative classification scheme is that proposed for Sydney shale by Pells et al (1978), under which the material at Glyde Point can be categorised as Class IV to Class V shale. According to the text book definition of Attewell and Farmer (1976), materials comprising soil / rock with a high clay content and a tendency towards fissility can also be grouped collectively under the term clay shale. Johnston (1991) reports a definition of "soft" rock material with overlapping boundaries between Unconfined Compressive Strength (UCS) values for clay and rock, based on International Society for Rock Mechanics (ISRM) classification, and describes several shortcomings of in situ and laboratory testing of materials which fall into this category.

Recovery greater than 100% was recorded on many of the drilling runs in the more shaly zones, and the rock material was therefore inferred to be composed of indurated expansive montmorillonitic clay particles. The swelling, common in weak shale, occurs because of the combined effect of unloading from overburden removal and the wetting caused by drilling resulting in volumetric expansion.

Blocking of the diamond drill bit waterways occurred several times in the deeper boreholes, typically in zones of transition between competent claystone and weathered or weaker shaly material. Drilling progress was slow as a result of the need to remove the drill rods from the hole in order to unblock bit waterways. To reduce the frequency of blocking, a relatively low bit pressure, high rate of rotation and high water pressure were maintained through most of the drilling. A face discharge bit was used initially but was replaced by a stepped bit, with side discharge waterways.

4.3 Groundwater

Standpipe piezometers consisting of 50 mm slotted PVC were installed in five of the boreholes, and groundwater samples recovered for laboratory testing, in accordance with British Standard BS 5930. Chemical analyses performed on the samples revealed low pH values of the order 3.0, which was initially suspected to indicate prevailing acid-sulphate soil conditions. Further sampling and testing revealed low sulphate (SO_4) concentrations, and it was later found that acidic groundwater is widespread in coastal areas of the Northern Territory, and occurs as a result of dissolved CO_2 originating from the high CO_2 respiration rate of tree and plant roots in aquifer recharge areas during the wet season (Marks and Jolly, 1987).

Acidic groundwater conditions would have an impact on foundation construction, particularly piled foundations. According to the American Concrete Institute (ACI) Guide to Durable Concrete (1992), a protective barrier system would be necessary to prevent the corrosion of concrete. Similarly, a protective barrier system would also be required to prevent the corrosion of steel piles.

5 IN SITU AND LABORATORY TESTING

The laboratory testing which is summarised in Table 1 below was carried out on selected core samples recovered from diamond drilling:

Test	Qty	Purpose
Moisture Content and Dry Density	55	Indirect determination of material strength and deformation parameters
Point Load Index	10	Indirect assessment of material strength
Unconfined Compression	33	Direct assessment of strength and indirect assessment of deformation parameters
Unconsolidated Undrained Multi-stage Triaxial	2	Direct assessment of strength and deformation parameters
Consolidated Undrained Multi-stage Triaxial	3	

Table 1: Laboratory Testing

Core sample selection in soil and rock materials was restricted by the minimum length of intact sample required for testing and fragmented nature of the core. Triaxial testing of soil materials was carried out under the presumption that confining pressures would reinstate the sample to a condition of integrity close to that of an undisturbed sample.

6 ANALYSIS OF DATA

6.1 Laboratory Test Data

The results of the laboratory testing showed that the soil strength was relatively high, and the rock material strength generally low. For example, the maximum measured UCS value was 2.7 MPa, recorded for a sample recovered at a depth of 57 m, and pocket penetrometer testing with an upper limit of 450 kPa was able to be carried out throughout the profile. Johnstone (1991) reports a definition of "soft" rock material with overlapping boundaries between UCS values for clay and rock, for strengths between 0.5 and 25 MPa, under which basis the materials can largely be termed "soft" rock. The UCS results are similar to those reported by Sales (1988), for the testing of the foundations of the bridges constructed to provide access to the Channel Island Power Station, at Elizabeth River and Channel Island. The UCS values in the upper 12 m of the profile ranged between 0.5 MPa and 2.5 MPa. By contrast, Clarke et al (1999) reported the range of compressive strengths for silicified material, or porcellanite, in the Darwin region as being between 2 and 160 MPa, which is significantly higher than for the silicified materials at Glyde Point. Strictly speaking, therefore, the materials at Glyde Point cannot be referred to as porcellanite.

For the materials at depth, a simple least squares linear regression analysis was performed to investigate the relationship between UCS and Young's Modulus, (E). Assuming the material behaviour to be like that of a cohesive soil, the undrained shear strength, (S_u,) was taken as half of the UCS value, and the ratio E/S_u compared with ratios published in the literature and available in standard soil mechanics texts. A relationship $E = 228\ S_u$ was derived, which is comparable with the relationship reported for overconsolidated clays by Bromham and Styles (1971), namely, $E = 250\ S_u$. The minimum plasticity index (PI) value for materials at Glyde Point was determined to be 32%, and Bowles (1997) quotes the relationship $E_s = (100$ to $500)\ S_u$, for materials which have a PI greater than 30.

A comparison was also made between the various other parameters, either measured or derived from field and laboratory data. The relationship between moisture content versus dry density was examined, and an inverse linear trend observed, and the relationship between moisture content versus E was examined, to determine whether a simple correlation could be made allowing E to be derived from moisture content tests, which are cheap and easy to perform, rather than unconfined compression and triaxial tests.

Correlations between E and UCS and moisture content exist for other similar geologic materials both in Darwin and elsewhere in Australia, and form the basis for an expected relationship between these parameters at Glyde Point. Nyland and Gerner (1984) presented a chart which shows moisture contents ranging from 4 to 24% and corresponding UCS values between 6 and 20 MPa, for BIF siltstone and claystone in the Darwin region. Ghafoori et al (1993) reported a relationship between moisture content and unconfined compressive strength for Ashfield Shale, from the Sydney Basin, in New South Wales, for a range of moisture contents between 1 and 9% and corresponding UCS values between 2 and 35 MPa, and Johnston (1992) reported the trend of saturation moisture content against secant modulus values for Melbourne mudstone, with moisture contents ranging from 3 to 16%, and modulus values in the range 100 and 6000 MPa.

The range of results at Glyde Point, for both E and UCS values was lower and moisture contents greater than for each of the three regions above. The results also showed a wide scatter and no clearly defined trend. The lack of a relationship between the measured parameters reflects the varying degrees of sample disturbance which inevitably occurred due to the drilling and sampling procedures which were used.

6.2 Standard Penetration Tests

Johnston (1991) has described several shortcomings of in situ and laboratory testing of materials which fall into the "soft" rock category, including mention of the fact that SPT is generally precluded because of the effort required to penetrate the materials.

At Glyde Point, a plot of the results of SPT versus depth for the upper 10 m revealed a wide scatter of results, reflecting varying degrees of cementing caused by lateritic weathering and / or silicification of the soil profile. The SPT N-values were therefore found to be most useful as a crude indicator to discriminate between materials behaving as either soil or as weak rock.

Look (1997) discusses the interpretation of SPT N-values when SPT refusal occurs. SPT refusal, defined in Australia as 30 blows for less than 100 mm penetration or hammer bounce on 5 consecutive blows, is typical in hard soils and weak rock. Equivalent N-values can be determined, although there is no standard procedure for doing so. The simplest method of determining equivalent N-values is to assume a linear increase in blows with depth. If the values obtained fall within the range of 80-200, as was typically encountered at Glyde Point, the result is considered to be characteristic of weak rock.

6.3 Point Load Testing

The limited number of point load test results Point Load Index (Is_{50}) were compared with UCS values and a relationship UCS = 13 Is_{50} found by linear regression, with a corresponding correlation coefficient of 0.55. The constant in this equation is about half of the value reported by Pells (1975), but is consistent with the relationship reported by Nyland and Gerner (1984). It can be concluded, however, that since there is a poor correlation between Is_{50} and UCS values, point load testing is an inappropriate testing procedure for predicting the compressive strength of material such as the clay shale encountered at Glyde Point. The test results, however, did confirm that the rock material strength is in the extremely low to very low range ($Is_{50} < 0.1$ MPa).

6.4 Rock Mass Rating

An assessment of the Rock Mass Rating (RMR) was made using the method established by Bieniawski (1989). RMR determination was deemed to be appropriate for rock mass characterisation as it takes into account the strength of intact material, RQD, discontinuity spacing, the condition of discontinuities, and the presence of groundwater. From the RMR values, E was derived using the Bieniawski formula. The RMR method, however, generally has applicability to higher strength rock masses. The derived moduli values were generally an order of magnitude higher than those established by laboratory methods, and were therefore considered to be an upper bound estimate.

6.5 Geotechnical Model

The end result of the interpretation of data was a seven-layer geotechnical model, comprising zones of relatively low strength surficial soil, higher strength indurated weak rock, low strength hard clay, weak interbedded clay shale and intact claystone, and finally higher strength massive claystone bedrock. Unique strength and deformation properties were identified for each zone, and as the properties based primarily on the results of the laboratory UCS and triaxial testing.

7 ALTERNATIVE TESTING METHODS

In light of difficulty experienced obtaining good quality samples for laboratory testing, in situ testing is generally the most suitable means of determining strength and stiffness parameters for detailed design. Typical in situ testing methods include surface seismic refraction surveying, crosshole seismic measurements, high range pressuremeter and self-boring pressuremeter testing (Ervin, 1983). The latter of these methods would be likely to produce the most reliable results, because the test procedure does not rely upon insertion of the testing equipment into a pre-bored hole. Pre-boring in low strength clay shales is likely to result in softening and possibly erosion of the sides of the hole and therefore adversely affect the testing results. Pile load testing is another routine procedure however is the most costly means of establishing design parameters.

Sales (1988) reports the results of pressuremeter and pile load tests carried out for the design of foundations for the Channel Island bridges. The ground conditions comprised completely to moderately weathered, sub-vertically bedded siltstone and phyllite. The results showed a backfigured rock modulus of twice the measured pressuremeter modulus under compressive loading, and approximately equal values under lateral loading. At Glyde Point, it is evident that since the clay shale is horizontally bedded, the modulus under lateral loading conditions will be higher than under compressive loading, and pressuremeter testing is therefore likely to underestimate the true range of rock mass modulus values for compressive pile loading conditions.

8 APPLICATION TO PILED FOUNDATIONS

8.1 Pile Types

As a part of the work carried out, an assessment and recommendations were made regarding suitable piling techniques. Due to the presence of alternating strata of hard soil and "soft" rock, there are significant disadvantages to driven pile footings:

- the near-surface cemented layer would be impenetrable to all except small solid section or tubular steel piles
- alternating bands of weak rock and hard clay would result in slow and erratic penetration rates. Although pre-boring would be possible it would be uneconomic to use a two-stage installation system to any significant depth
- the high level of energy required to drive piles would significantly disturb weak, brittle rock, and would reduce rather than increase the capacity gained by side adhesion.

Bored cast-in-place piles, on the other hand, were assessed as having several advantages:

- more reliable adhesion because of lesser disturbance
- the larger pile diameter which can be used provides a significant end-bearing component
- larger diameter piles enable inspection of the base and adjustment to the founding depth is not costly

Rock socketed piles were used in construction of foundations for the Elizabeth River and Channel Island bridges (Sales, 1988).

8.2 Pile Design

Preliminary design calculations were carried out to determine pile capacities, using parameters from the geotechnical model to derive ultimate shaft adhesion and end bearing resistance values. Ultimate fully mobilised adhesion for stiff to hard clays in the literature (Poulos & Davis, 1989; Winterkorn & Fang, 1975) is recommended as not greater than 0.4 - 0.5 S_u for both driven and bored piles, and the geotechnical strength reduction factor in AS 2159 - 1995 is taken as 0.45. With an average load factor of 1.35, this is equivalent to a factor of safety of 3. For brittle soil conditions, such as at Glyde Point, a skin friction of slightly less than 0.4 S_u was recommended. The ultimate end bearing resistance was calculated as 4.5 times the unconfined compression strength, consistent with 9 times the undrained cohesion for clays, as recommended by Poulos (1989) and AS 2159. Since very little definition was given of the actual design loading conditions, the calculations performed were limited to prediction of pile capacities for only two simple example pile diameters and lengths, namely piles of 0.5 m and 1.5 m diameter and 15 m and 40 m length, with predicted ultimate capacities of 2,500 kN and 22,000 kN respectively.

9 CONCLUSIONS

A geotechnical investigation has been carried out as part of the feasibility study for a proposed LNG processing facility at Glyde Point, near Darwin, in the Northern Territory. The local geology comprises material which can be described as lateritised clay shale, forming part of a geological unit named the Wangarlu Mudstone Member, within the Bathurst Island Formation.

The nature of the material encountered is similar to that in the Darwin city area. The material strength is low and typical of a group of materials which can be referred to as "soft" rock. Conventional investigation sampling and testing techniques, namely SPT and "undisturbed" thin-walled tube sampling are generally inadequate, as the material strength is too high both for meaningful results to be obtained and for samples to be recovered.

Good core recovery can be obtained from diamond drilling operations, however sampling disturbance is difficult to avoid due to low material strength, wetting of lateritised material and remoulding and swelling during drilling of shaly material, and mechanical disturbance during removal from the inner core barrel.

In situ pressuremeter testing is recommended as a more suitable method of determining elastic deformation parameters, and combined with the results of unconfined compressive strength testing is considered likely to be the most suitable method for obtaining parameters for design of piled foundations.

10 ACKNOWLEDGEMENTS

The author wishes to thank Woodside Energy Ltd for giving their kind permission to publish the paper, and senior geotechnical engineering staff at GHD for their assistance.

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