Geotechnical Design of an Offshore Gravity Base Structure

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ABSTRACT

The paper focuses on the geotechnical design issues facing the design team responsible for the provision of an offshore Gravity Base Structure (GBS) to act as a clump weight for a Power Buoy located in the Lyell field of the northern North Sea, United Kingdom.
The structure is to be positioned on the seabed, where ground conditions are considered variable but in general comprise of a surface layer of loose to very dense silty sand, underlain by a thick sequence of firm to very stiff sandy clay. Geotechnical data specific to the location of the GBS is limited; problems encountered during testing allowed only three cone penetration tests to be carried out to maximum depth of 12m and sampling for subsequent laboratory testing was not possible. However, correlation between the results of these tests and CPT results for other areas in the North Sea, allowed geotechnical properties to be inferred and to be used as the basis for the geotechnical design of the foundation.

Assessment of the bearing capacity of the structure under hydrodynamic loading, as well as the resistance to sliding indicated that there maybe a risk of instability. As a consequence, a perimeter skirt was specified in order to reduce this risk. A discussion of the geotechnical issues considered during the design process is presented. Other design issues such as cyclic loading and penetration resistance in relation to a perimeter skirt, were considered and are commented on within this paper.
The case history highlights some of the design problems faced by geotechnical engineers when designing structures for the offshore environment, and emphasizes the necessity for a comprehensive and, site specific, ground investigation, in order to facilitate the design process.

INTRODUCTION

Offshore foundations are subjected to complex forces generated by a combination of externally applied man-made forces, the structural weight and environmental loads from current and wave forces. This complex combination of loading conditions applied to the foundation may result in overturning moments leading to uplift forces, as well as downward compressive loads. Additionally, the combination of loadings will inevitably lead to the development of a component of loading acting parallel to the seafloor. As a consequence, a shallow foundation located in an offshore environment must be designed to resist downward-bearing forces, horizontal forces and upward forces, depending upon the loading arrangement. Design methods, informed by relevant codes of practice as well as fundamental soil mechanics principles, are considered and documented in the following sections, with specific reference to a proposed gravity base structure (GBS) to be located on the sea bed in the Lyell Field of the northern North Sea area, immediately West of the Ninian Field.

The gravity base serves as a clump weight and manifold for a 100m high Power Buoy. The location of the study area is indicated in Fig. 1.

The primary focus for the study documented herein is related to the geotechnical assessment of the interaction of the gravity base with the known or assumed seabed strata based on geological and geotechnical data. Specifically, the work comprises of:

- an assessment of the proposed gravity base in terms of its resistance to sliding during storm conditions, when subjected to uplift and lateral forces;
- an assessment of the proposed gravity base in terms of its bearing capacity;
- an assessment of the effect of a perimeter skirt located on the under-side of the gravity base in order to increase lateral resistance by mobilizing the strength of deeper, more competent soil layers; and
- an analysis of the ability of the skirt to penetrate the seabed to the required depth.

In carrying out an assessment of the stability of the GBS, a number of important issues were identified and are highlighted in this paper.

GEOLOGICAL SETTING

RPS Energy (RPSE) were commissioned by Canadian Natural Resources (UK) Limited (CNR) to carry out a data integration study in 2006, which sought to optimize the existing geological and geotechnical data for the shallow soils at the proposed GBS location. The study used geotechnical and geophysical data from the region in combination with British Geological Survey information in order to develop an appropriate geological and geotechnical model for the site.

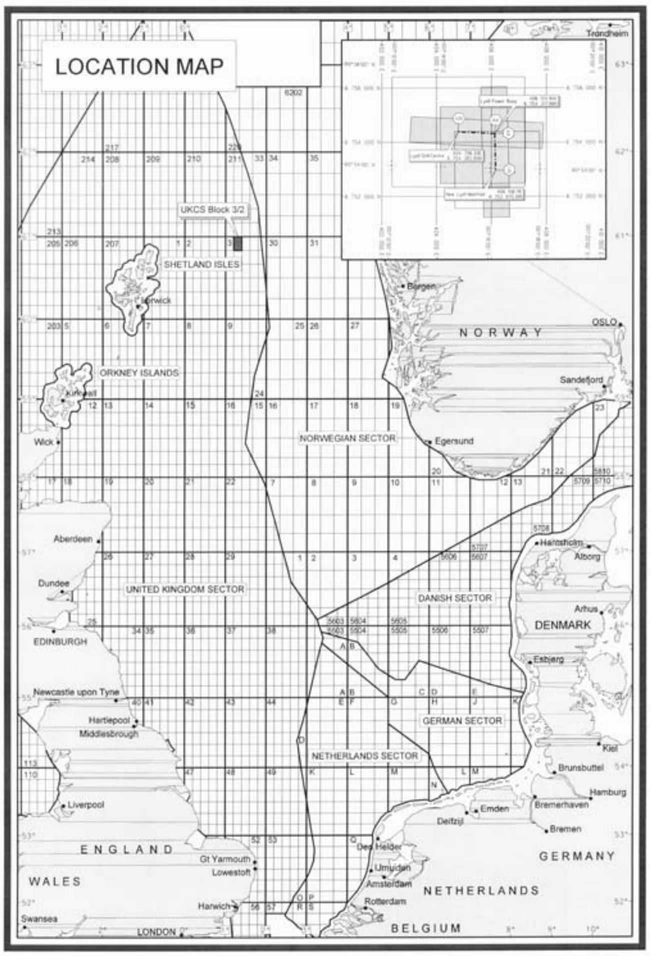

Fig. 1. Location plan

A significant amount of information has been produced for this region, much of which relates to the development of the Ninian and the Lyell fields to the south-east and south of the study area, respectively. This data includes detailed bathymetric, seabed and geological information, as well as geological and geotechnical data from borehole investigations undertaken by Fugro Survey Limited (and Fugro McClelland). This data concerns investigations carried out over a 30 year study period, and includes borehole logs to depths between 40 to 150m below the seabed, and CPT data to a depth of 30m. It can therefore be considered that the soils in this region have been extensively studied.

Between February and March 2007, geotechnical investigations were carried out at the proposed GBS location by UTEC, and took the form of three Cone Penetration Tests (CPT). However, due to operational difficulties, the CPT's were only completed to a maximum depth of 13m below the surface of the seabed; two of the three tests only achieved a depth of circa 4m.

In the absence of site specific information, soil sampling borehole data and CPT data for other locations in the region were combined with this data to give an indication of the anticipated near-surface geology at the proposed location. Table 1 summarises the soil profile at the GBS location.

Table 1. Soil profile at GBS location

Soil Unit	Depth below seabed (m)	Geological Formation	Soil Description
A	0.0 – 0.6	Holocene	Loose, fine to medium SAND with shell fragments
B1	0.6 – 6.5	Ferder	Firm to very hard sandy CLAY with very dense sand layers and occasional gravel
B2	6.5 – 39.0		Very stiff to very hard, sandy CLAY with occasional gravel and dense to very dense SAND layer at 29 to 32m
C	39.0 - >50.0	Mariner	Dense silty SAND

DESIGN CONSIDERATIONS

<u>Gravity Base Structure</u>

The GBS is in effect a clump weight imparting a vertical load on the surface of the seabed. However, since it is attached to a Power Buoy by tethers, acting in tension, such that the Power Buoy is maintained at a constant depth, the GBS would be subjected to vertical uplift forces. Additionally, due to hydrodynamic forces (wave loading and currents), the GBS would also be subject to a potentially significant overturning moment.

As a consequence, the GBS should be designed to have sufficient mass to resist the uplift forces, but clearly the submerged mass should not exceed the bearing capacity of the seabed soils. The overturning moments will generate an eccentricity in the loading conditions, and this must also be accounted for in the design, as this will reduce the effective area of the foundation, and therefore increase bearing stresses. Similarly, wave and current loadings will generate a significant lateral component to the loads applied to the seabed, and this should also be allowed for in the analysis. This inclination of the resultant load will change the form of the bearing capacity failure surface, allowing bearing capacity failure to occur at lower loads.

The lateral loading component may also induce a sliding failure at the interface between the lower face of the GBS and the seabed soils. The resistance to sliding is affected by the shear strength of the soils and by the forces acting normal to the anticipated failure surface (under drained conditions), as well as the area over which the shear strength is mobilized, i.e. the footprint of the GBS (for undrained conditions).

For any potential site for the location of a foundation, specific information is required and would normally be obtained through a clearly defined site and ground investigation. Guidance for the content and scope of such investigations is provided in a number of documents; for onshore geotechnical design in the UK, guidance is provided by BS5930 (1999) and more recently, Eurocode 7 (2004). For offshore design, additional guidance can be sought from DNV Classification Note 30.4 (1992), as well as other documents, such as OFT 2001/014 (HSE, 2001), BS EN ISO 19901-4 (BSI, 2003) and the Handbook for Marine Geotechnical Engineering (NCEL, 1985).

From a geotechnical perspective, the information obtained from the site investigation should allow the designer to predict with confidence the behaviour of the soils under the anticipated loading conditions, and the design of the investigation, should therefore, be informed by, and be driven by the geotechnical design engineer.

In this case, the site investigation was conducted under the instruction of CNR, who would be the end-users of the GBS and the attached Power Buoy, with minimal input from the design engineers. As a consequence, the investigation, as suggested in the previous section, was inadequate.

In summary, the investigation involved a desk study, to collect and collate existing data from a wide range of sources over a wide geographical area, and use this in combination with three (number) CPT boreholes driven across the anticipated footprint of the GBS foundation, in order to infer relevant geotechnical design properties. The maximum depth achieved with the CPT's was 13m below the seabed, with two of the three only achieving a maximum penetration of 4m.

At the proposed site for the GBS, no further testing was undertaken, no sampling was attempted, and therefore no site specific geotechnical properties were determined, other than those that could be inferred from the results of the limited CPT test data. Fig. 2 shows the measured cone tip resistance with depth for the three cone penetration tests carried out at the site, but also shows data for two further locations, the Ninian North platform and the Lyell Manifold, 16km south-east and 1.3km south of the proposed GBS site, respectively. Fig. 3 shows the measured sleeve friction versus depth for the same locations. Additionally, Fig. 12 shows the complete CPT data beneath the centre–point of the proposed foundation.

<u>Geotechnical Investigation</u>

Geotechnical profiles through the soil sequences for the proposed Power Buoy location is limited to Cone Penetration Test (CPT) data, however, these CPT profiles have been conducted to a depth of only 13m (Fig. 2 and Fig. 3). Below this, the soil profile prediction is based on geophysical data only. Correlations have been drawn between this area and borehole data for the location of the Lyell Manifold and Ninian Platforms (1.3km South and 16km South-East of the proposed Power Buoy location, respectively), and supporting and consistent data sets for the Ninian Central and Southern platforms have also been examined.

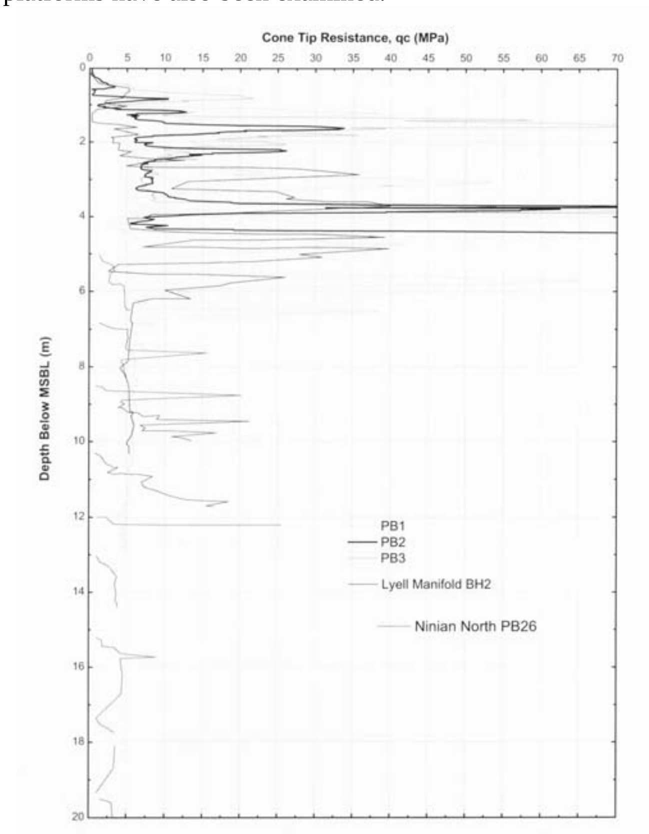

Fig. 2. Measured cone tip resistance q_c versus depth

It is clear that this investigation is inadequate and that very little data has been directly obtained from the proposed Power Buoy location. Laboratory test data has been provided by RPSE (2006), which includes:

- Undrained shear strength;
- Atterberg Limits;
- Soil stress history;
- Coefficient of volume compressibility; and
- Internal angle of friction

A value for the Poisson's ratio is also quoted, as is a relationship between shear strength and the Young's modulus (RPSE, 2006). However, it should be emphasised this data has been obtained from sites other than the proposed Power Buoy location. The geotechnical data sets have been correlated from a number of different locations; the existing Lyell Manifold located 1.3km South of the proposed Power Buoy location, three Ninian Platform sites, plus information from two further alternative sites. The predicted soil profile from these data-sets, however, *represents a conservative assessment for the purposes of foundation design* (RPSE, 2006).

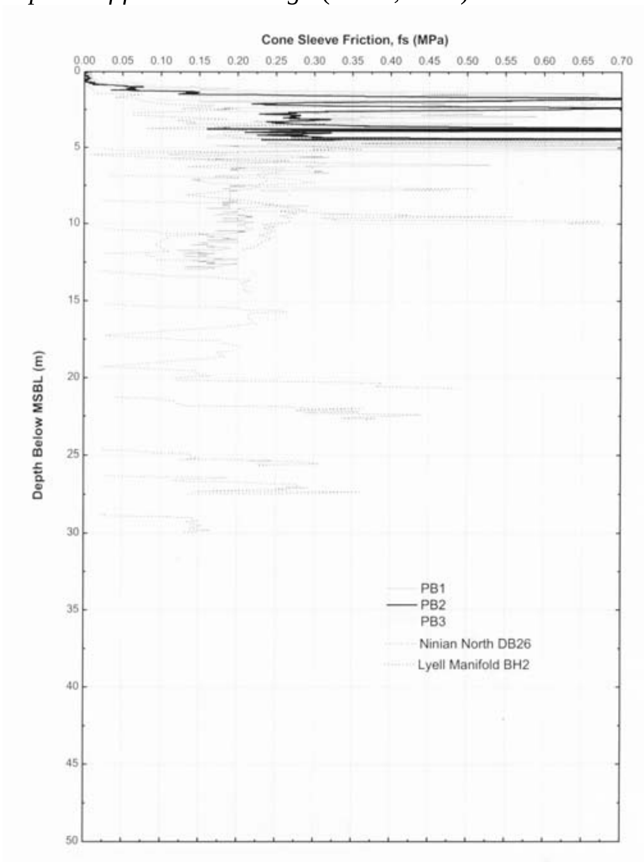

Fig. .3 Measured cone sleeve friction versus depth

Sufficient guidance on what is required as part of an offshore geotechnical investigation is widely available, and it was strongly advised by the authors that adherence to these standards is required to enable appropriate soil parameters to be determined to ensure short and long term geotechnical stability of the proposed Power Buoy gravity base. It is considered here that although there is evidence to suggest that the correlations referred to are perhaps valid, foundation design of structures of such engineering significance cannot be undertaken with the necessary degree of confidence without an adequate, site specific ground investigation. In this instance, this has not been undertaken.

GEOTECHNICAL PARAMETERS

Introduction

The following sections present the geotechnical parameters interpreted from the CPT datasets correlated against data obtained from other areas of the North Sea region.

Atterberg Limits

Fig. 4 shows the relationship between Liquidity Index (LI) and depth. The data indicates that the majority of soils in the upper 20m have LI values between ±0.5, suggesting that the soils are at or near to their plastic limit. It is possible to calculate the remoulded strength of the soil based on the Atterberg Limits (eg. Skempton & Northey, 1953; Schofield & Wroth, 1978).

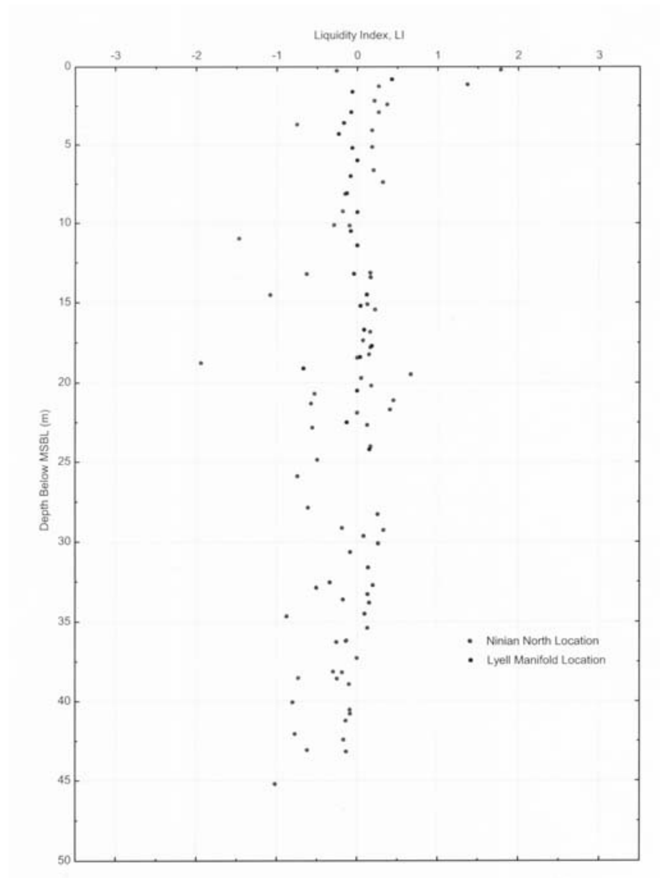

Fig. 4. Liquidity index versus depth

Fig. 5 shows that the majority of soils are clays of predominantly low to intermediate plasticity.

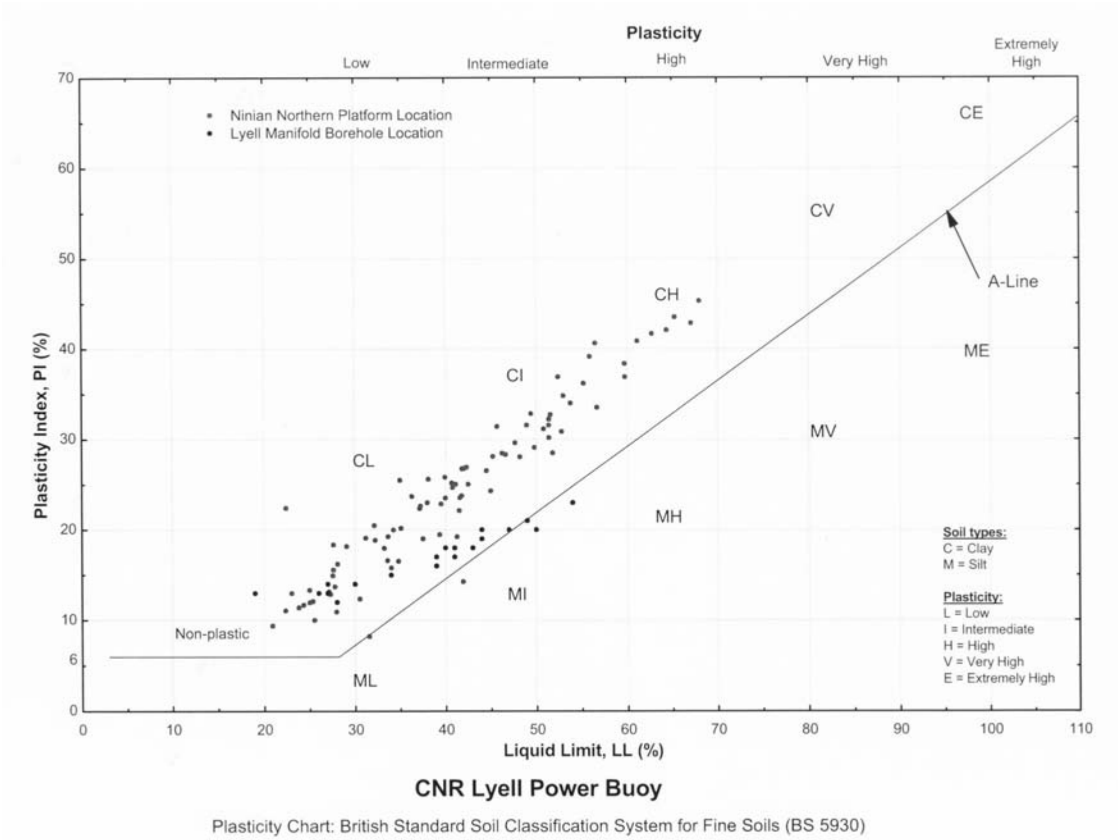

Fig. 5. Plasticity characteristics

Soil Stress History

The stress history of the soils has been examined in order to support the attempt to correlate the CPT data at the proposed GBS site with data obtained for soils elsewhere in the North Sea region. The following empirical relationship was used to relate the Over-consolidation Ratio (OCR) of a soil to the CPT data (Ladd et al, 1979):

$$OCR = \left[\frac{\left(s_u / p'_0 \right)_{OC}}{\left(s_u / p'_0 \right)_{NC}} \right]^{\frac{1}{\lambda}} \qquad (1)$$

In which, s_u is the undrained shear strength; p'_0 is the effective overburden pressure; λ is the ratio of swelling to compression indices taken from oedometer tests, and in this case assumed to be 0.8; and the subscripts OC and NC refer to over-consolidated and normally consolidated, soils, respectively.

The results of this analysis are presented in Fig. 6, which shows a reasonable correlation between the derived OCR data and that correlated from the GBS CPT data, in terms of trend, although there is an offset in the CPT inferred data. The results indicate that the soils from the Ninian North platform location and the Lyell Manifold location have a similar stress history to those at the GBS site. It was considered therefore, that soil properties inferred from these datasets, would be appropriate for design.

Undrained Shear Strength

The undrained shear strength of the soils from all locations, based on laboratory tests is presented in Fig. 7. Inferred shear strength profiles for the GBS location, based on the CPT test results were compared with these datasets, and it was found that the inferred strength lies within the overall range of the laboratory dataset. However, the dataset (Fig. 7) shows a very wide spread of data points, as a consequence, a best estimate shear strength profile was suggested, based on the lower bound laboratory test data, which was:

0.0m to 10m: s_u = 25kPa + 22.5kPa/m
10m to 15m: s_u = 250kPa + 25kPa/m
15m to 30m: s_u = 125kPa
>30m: s_u = 250kPa

Compressibility Characteristics

Values for the coefficient of volume compressibility (m_v) inferred from Oedometer test data (Fig. 8) for stress range

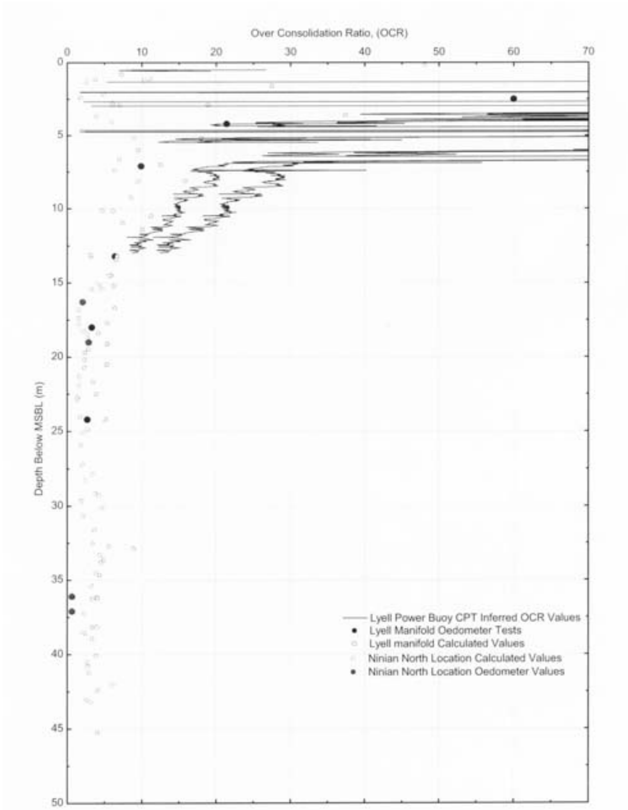

Fig. 6. Over-consolidation ratio versus depth

100kPa to 300kPa, indicates values of between $0.1m^2$/MN and $0.02m^2$/MN.

Since the soils are saturated, it is appropriate to assume a value for the Poisson's ratio (ν) of 0.5. The undrained Young's Modulus may be inferred from the undrained shear strength profile.

Drained Shear Strength

Again, laboratory test data from other locations has been correlated against inferred data from the CPT's. These are shown in Fig. 9, which indicates a wide range of values for the upper sediments (<5m). However, ignoring the loose material at the surface, a lower bound value of 32deg might be assumed for design.

Cyclic Shear Strength

The static undrained shear strength may be used for cases where the governing load has a mainly static character. The effects of cyclic (wave) loading on the shear strength should also be considered in other cases. Cyclic loading may cause pore water pressures to build up in the soil possibly leading to a reduction in shear strength.

The undrained shear strength under cyclic loading can be defined in one of two ways:

- reduced static shear strength to reflect the effects of cyclic loading; or
- cyclic shear strength defined as the sum of the static and cyclic stress that induces failure for a given number of cycles.

Cyclic loading can affect the static material shear strength in two ways; during a storm, the loading rate applied to the soils is very quick, as a consequence, there can be a significant increase in the undrained shear strength. However, as a result of repeated loading/unloading cycles during a storm event, the undrained shear strength will decrease. The determination of the cyclic shear strength is considered to be the most appropriate way of accounting for both the loading rate effects and the cyclic degradation.

The cyclic shear strength is related to the static undrained shear strength by the cyclic load factor, U_{cy}, values of which are site specific and should be determined from appropriate laboratory tests on soils specimens from the actual site. In this case, since no samples were obtained, this phenomenon could not be examined.

It is however, possible to compare the soils at the proposed GBS location with those for other sites in the North Sea for which cyclic shear strength data is available, and use this data to establish a cyclic shear strength for the soils at the GBS site. In this case, particle size distribution data was utilised to develop this correlation, and a cyclic shear strength was subsequently inferred.

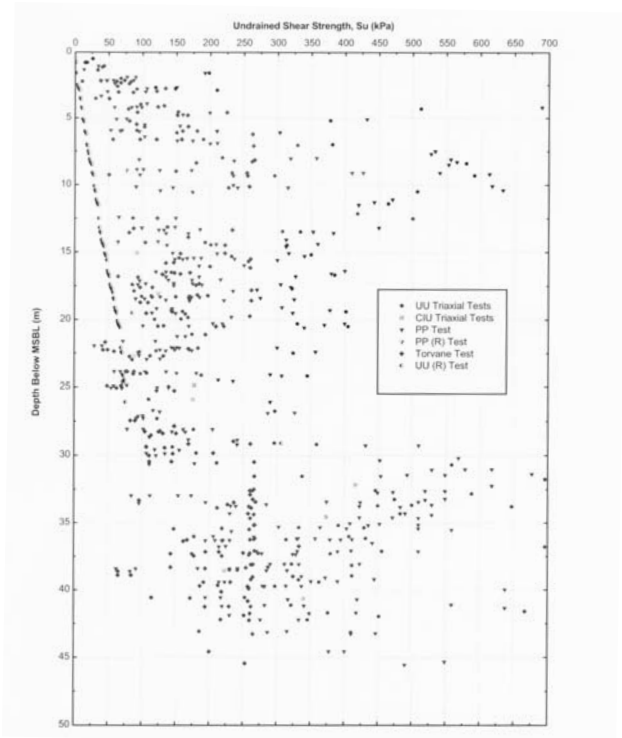

Fig. 7. Undrained shear strength versus depth

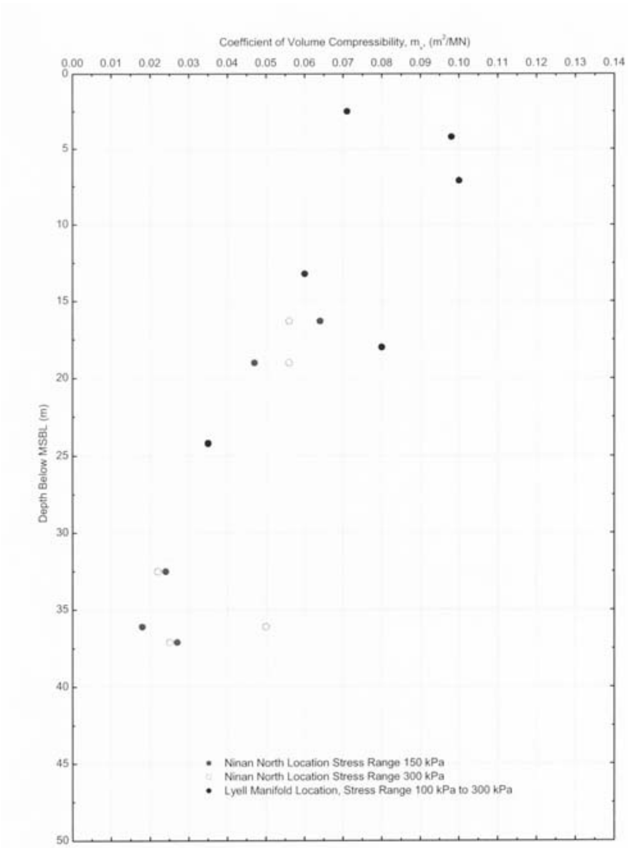

Fig. 8. Compressibility versus depth

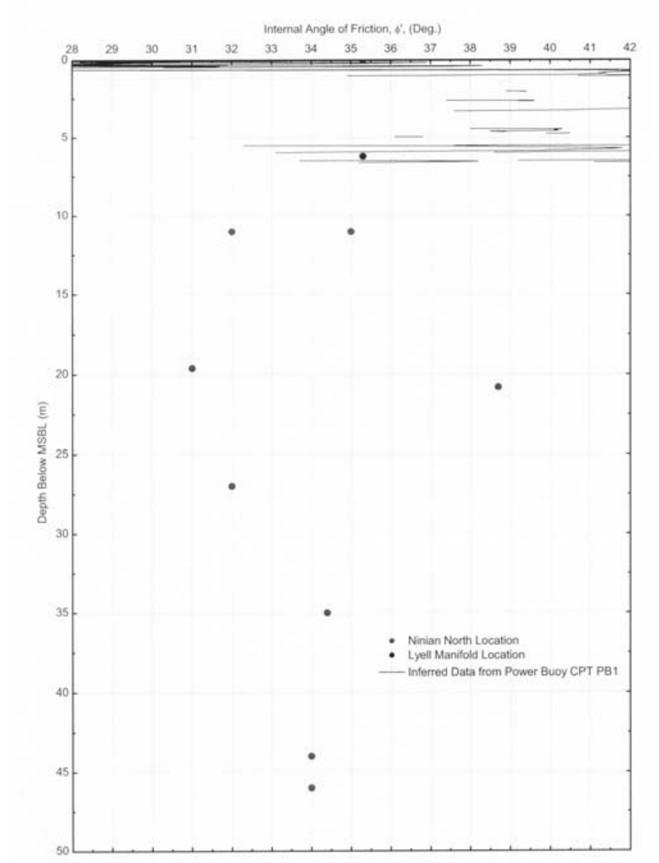

Fig. 9. Effective angle of friction versus depth

GEOTECHNICAL ANALYSIS

<u>Introduction</u>

Construction of the GBS clump weight began prior to the involvement of the primary author of this paper, and prior to any significant geotechnical design input. Some preliminary assessment of the stability of the structure was undertaken, but this assessment was limited. The implications of this could be quite significant. If an initial geotechnical assessment was carried out based on the geotechnical parameters specified in the previous section, in the context of bearing capacity and resistance to sliding, using the as-built geometry of the GBS, was to show that stability was an issue, than any suggested modifications to the design would have to be retrofitted, with significant cost implications.

The overall design of the GBS foundation covered the following specific items:

- Foundation instability – bearing capacity and sliding resistance under undrained, drained and cyclic loading conditions (by adopting a cyclic shear strength for the soils); and
- Skirt penetration resistance.

Initial Design

The initial design for the GBS clump weight was based on a honeycomb concrete structure with a footprint of 20m by 20m. The honeycomb structure would have two functions, the first was to provide space for mechanical equipment (pumps, manifold etc), and the second to provide space to add ballast if required during submersion and emplacement. Detailed hydrodynamic analysis carried out by Monitor Oil, provided loads that could be used as part of the design. Table 2 summarises the loads applied within the geotechnical analysis. The table indicates that an upper and lower bound estimate was made of the submerged weight of the GBS.

Fig. 10 shows the final structure, but indicates that the GBS was constructed with 4 bearing pads located at each corner; these have a bearing area of 4.5m by 4.5m each.

Table 2 Design loads

	Unfactored	Factored (1.3)
Actual Uplift	9287kN	12073kN
Actual submerged weight	16672kN	-
	17496kN	-
Actual horizontal load	1108kN	1440kN
Actual vertical load	7385kN	5681kN
	8209kN	6315kN

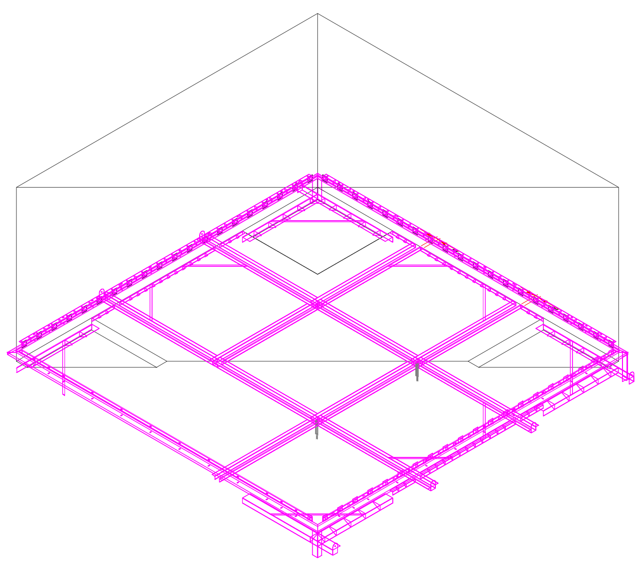

Fig. 10. Section through GBS clump weight, showing skirted foundations

Final Design

A stability assessment of the initial design, as-built, indicated that for the ultimate limit state condition, in terms of bearing capacity and settlement, the foundation was satisfactory. However, in terms of sliding resistance, the stability assessment indicated that the limit state was exceeded. As a consequence, the design was modified, retrospectively. In this case, it was decided that the lateral load capacity could be increased by incorporating shear keys or a perimeter skirt on the base of the GBS (Fig. 10). The intention was to force the failure surface, which for the initial design was located at the interface between the base of the structure and the soils, deeper in to the seafloor soils, to mobilize the higher shear strength of the soils at this depth. In order to minimize costs, the length of the skirt was limited to 1m around the perimeter, with 0.5m long skirts approximately 6.5m apart across the base of the GBS.

The resistance afforded by the perimeter skirt is determined by the depth of penetration (related to the length of the skirt, the submerged weight of the GBS and the resistance of the soils) and the horizontal distance between skirts, as these factors determine the mode of failure of the foundation. Lam et al (1987) showed through numerical modelling that, a skirted foundation can fail in a number of modes as shown in Fig. 11.

Generally, the skirts should be designed to be sufficiently close together to force the critical failure surface along the 'tips' of the skirt. The design procedure is detailed in NCEL (1985) and DNV (1992). Due to construction constraints, the horizontal distance between the skirts could not be optimized to ensure a tip-to-tip failure mode, therefore, the resistance to sliding of the skirted foundation, as-built was assessed. From Fig. 11 it is clear that the intention of forcing a tip-to-tip failure mode is to ensure that the shear resistance of the more competent strata at the tip of the skirt is fully mobilized. If this mode of failure cannot be ensured, than the critical failure surface will pass from through the weaker, shallower soils. An assessment was therefore made based on the assumption that the critical failure surface would be in accordance with Fig. 11 (c).

Using the undrained shear strength profile discussed earlier, the resistance to sliding was assessed whilst incorporating the effects of the passive wedge behind the skirts. This indicated this limit state was not exceeded.

The final element to be considered from a design perspective is the possible resistance to penetration of the skirt in to the seabed soils. DNV (1992) outlines the procedure for estimating skirt penetration resistance using CPT data. This approach requires that:

1. Identify the soil strata from soil borings and CPT's;
2. Determine for each CPT an average cone penetration resistance, $q_{c,av}$ at even intervals, eg. 0.2m; and
3. Determine for each depth an average cone penetration resistance, termed q_c of a selected number of individual $q_{c,av}$ representing certain identified strata

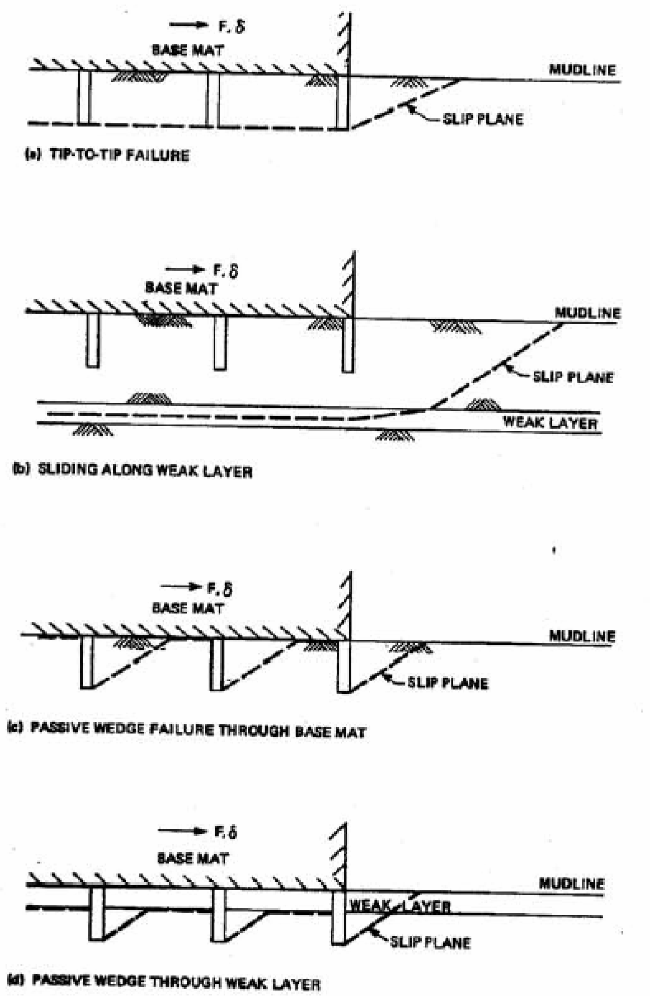

Fig. 11. Failure modes of skirted foundations (Lam et al, 1987)

The penetration resistance is then calculated from:

$$R = k_p(d)A_p q_c(d) + A_s \int_0^d k_f(z) q_c(z) dz \qquad [2]$$

In which:

d = depth of tip of penetrating member, m
$k_p(z)$ = empirical coefficient relating q_c to end resistance
$k_f(z)$ = empirical coefficient relating q_c to skin friction
$q_c(z)$ = average cone resistance at depth z, MPa
A_p = tip area of penetrating member, m^2; and
A_s = side area of penetrating member, per unit depth, m^2/m

Analysis is undertaken for the most probable penetration resistance and the highest likely penetration resistance. For this analysis, values for the empirical coefficients were taken directly from DNV (1992), and the cone penetration resistance was taken from a representative CPT, in this case CPT3 was carried out beneath the centre of the proposed GBS location, and was therefore deemed to be the most representative. This is shown in Fig. 12. The calculated resistance is then compared with the submerged weight to derive a factor of safety. In this case, under all loading conditions, penetration to a minimum depth of 1m was assured.

CONCLUSIONS

For typical offshore structures exposed to waves and currents, the underlying foundation soils must contend with static, dynamic, and impact force loads (actions). Static loads (permanent actions) are caused by the structure and foundation self-weight, and in most cases, these forces are relatively constant over the life of the structure. It is important to remember that buoyancy effectively reduces the weight of that portion of a structure beneath the water surface. Consequently, the structure self-weight load on the foundation soil will vary with tide elevation. In this instance, specific design elements within the foundation will reduce the effects of tide elevation on the loading conditions.

A structure's weight distribution and the differential loading applied to the foundation must be evaluated, particularly for gravity-type structures extending into greater depths or spanning different soil types. Lateral forces due to imbalanced hydrostatic pressure must also be considered.

Waves, currents, tides, storm surges, and wind are the primary dynamic forces (variable actions) acting on offshore structures, however, in some regions of the world earthquake ground motions may also induce severe dynamic loads. Dynamic loads vary greatly in time, duration, and intensity, and the worst likely load combinations should be examined during foundation design.

Using the limited geotechnical data available for the location of a 20m by 20m by 7m high gravity base structure, combined with more extensive data-sets for other regions in the North Sea, a modified design has been put forward by the authors for a structure that had previously been unfit for purpose. This modified design required the retrofitting of a perimeter skirt, to allow mobilisation of deeper soils with higher shear strength characteristics. The geotechnical assessment of this design has covered bearing capacity under vertical and non-vertical loads, sliding resistance under lateral loading conditions, as well as the effects of cyclic loading on the undrained shear strength of the soils. Finally, an assessment of the resistance to penetration of the perimeter skirt has been conducted to ensure that full penetration can be achieved.

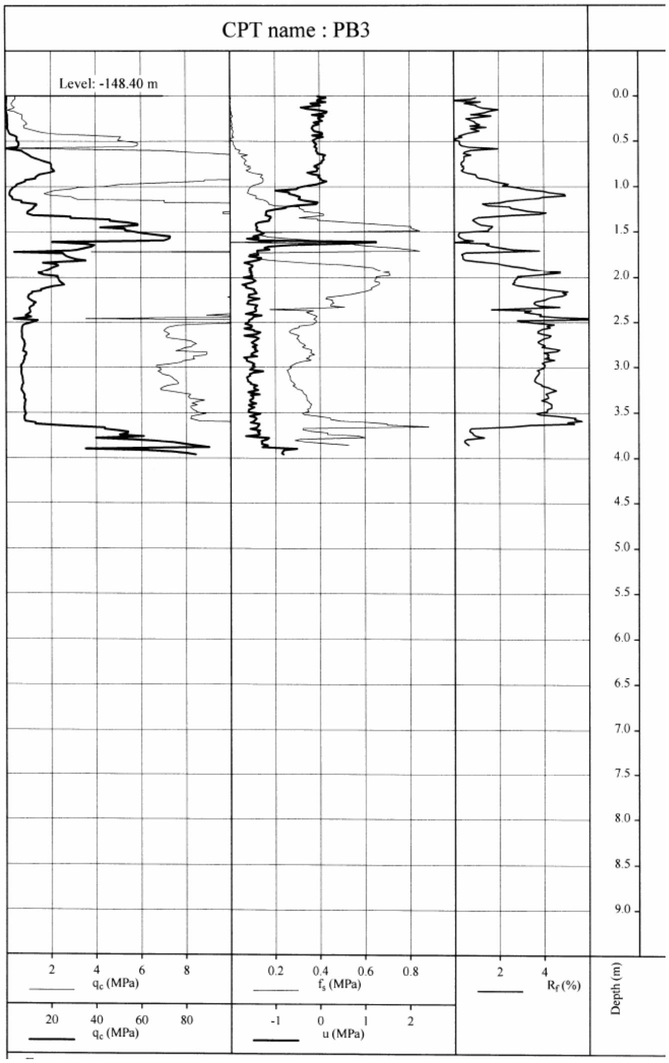

Fig. 12. CPT data used to determine penetration resistance

With limited site specific geotechnical data, a suitable offshore foundation design has been provided by back analysing and correlating this limited data with other adjacent sites for which more comprehensive information is available.

REFERENCES

British Standards Institution [1999]. *BS 5930: 1999 Code of Practice for Site Investigations*. BSI London.

British Standards Institution [2003]. *BS EN ISO 19901-4: 2003 Petroleum and natural gas industries – Specific requirements for offshore structures - Part 4: Geotechnical and foundation design considerations*, BSI London.

British Standards Institution [2004]. *BS EN 1997-1: 2004: Eurocode 7 Geotechnical Design – Part 1: General rules*, BSI London.

Det Norske Veritas [1992]. *DNV Classification Note No. 30.4 Foundations*.

Health and Safety Executive [2001]. *Offshore Technology Report – Foundations*, Report No. 2001/014.

Ladd, C. C., Foot, R., Ishihara, K., Schlosser, F. & Poulos, H. G. [1977]. *Stress-deformation and strength characteristics: State of the Art Report*. Proc. 9th Int. Conf. Soil Mech., Tokyo, Vol. 2.

Lam, I. Pelletier, J.H., Murphy, B.S., Sgouros, G. & Cheang, L.C. [1987]. *Design for lateral skirt resistance of arctic gravity platforms*. Proc. of the 6th Int. Conf. on Offshore Mech. and Arctic Eng., Houston, Texas.

National Civil Engineering Laboratory [1985]. *Handbook for Marine Geotechnical Engineering*. NCEL Port Hueneme, CA: US Naval Civil Engineering Laboratory.

RPSE. [2006]. *Proposed Lyell Power Buoy Tether Clump Weight, Data Integration Study,* Report No. EOG 0530-R-07-00.

Schofield, A. N. & Wroth, C. P. [1978]. *The correlation of index properties with some basic properties of soils*, Canadian Geotechnical Journal, 15(2), May, pp137-145.

Skempton, A. W. & Northey, R. D. [1953]. *The sensitivity of clays*. Geotechnqiue, 3(1), pp30-53.