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GEOTECHNICAL CHARACTERIZATION OF RIO CARIBE SOILS

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ABSTRACT

The results of a geotechnical study for the proposed Río Caribe offshore development are presented. The profile essentially consists of firm to very hard clays, with a surface layer of calcareous sand. Backanalyses of spudcan penetration at a site close to the site under study were used to evaluate engineering parameters for the carbonate sands. In addition, analyses of the resistance of the surface sands was also evaluated based on the results of several in situ and laboratory tests. A discussion of the problems of evaluating liquefaction susceptibility in calcareous sands is presented, principally in terms of shortcomings of the offshore SPT procedures employed.

INTRODUCTION

As part of the preliminary preparations for the engineering analysis, design and planning for the development of the Río Caribe offshore platform, a 500 ft (152 m) geotechnical boring was completed at the proposed location of the structure.

Both in situ and laboratory test data were obtained for the purpose of characterizing the geotechnical parameters of the foundation soils[1]. The objective of the field and laboratory testing program was to provide information for evaluating the foundation bearing capacity for piles under axial and lateral loads for static, cyclic and dynamic loading.

The offshore geotechnical boring and laboratory testing program were performed by Fugro-McClelland under contract to LAGOVEN, S.A. The field work was supervised by INTEVEP, S.A. who also performed corroboratory laboratory testing.

The purpose of this paper is to review the data available from the field and laboratory testing program and to present the engineering parameters that were used for the static, cyclic and dynamic foundation analyses. The data are taken from the various Fugro-McClelland field and laboratory reports produced for LAGOVEN and from the INTEVEP laboratory test results.

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angle of $\O_p$=43 degrees was obtained, with a steady-state value $\O_u$ of 20 to 25 degrees. The appropriate design value is notoriously difficult to determine for this type of soil. Experience in the Australian offshore environment is testimony to the difficulties associated in determining design values in these materials. In order to obtain an estimate of the friction angle for spud can capacity and penetration, a re-evaluation of field performance data was carried out for a site close to the Río Caribe area.

Determination of mobilized friction angle

A jack-up rig was used to drill an exploratory well at a location approximately 700 m from the Río Caribe site, where the surface soils are similar. The jack-up platform used was the Rowan Odessa. For a leg-load of 6 000 kips, the reported leg penetrations were:

Port leg:	11 ft (3.35 m)
Starboard leg:	16 ft (4.88 m)
Bow leg:	13 ft (3.96 m)

An initial estimate of leg penetration based on preliminary soil parameters was performed earlier and, for a leg load of 8 735 kips (38 853 kN) the penetration was estimated to be between 3 m (10 ft) and 5 m (16 ft).

While these predicted leg penetrations are in reasonably good agreement with the measurements (taking into account the difference in leg loads), it was decided to re-evaluate the field measurements in light of more detailed soil information that had been obtained from the soil boring results.

Two options are available for calculating the bearing capacity of spudcans, namely:

- standard bearing capacity theory
- cavity expansion theory

For the estimates given above, standard bearing capacity theory was employed, initially using a friction angle of 30°. N_q and N_γ values of 20 and 16, respectively, were obtained, which were reduced to 16 and 8 to account for the higher compressibility of calcareous sands. However, the reduction is arbitrary in nature as little information exists for confirmation.

To provide a lower bound estimate of the bearing capacity, a friction angle of 20°, which corresponds to the steady-state value, was selected (N_q = 15 and N_γ = 6.4).

In order to account for the higher compressibility of calcareous sands as compared to the more cammon silica sands, Terzaghi applied a correction to estimate the mobilized friction angle, $\O^*$, for calcareous sands according to:

$$\O^* = \tan^{-1}(0.67 * \tan\O_p) \quad (5)$$

Using a friction angle of 30° for the Río Caribe sands, this reduction suggests a mobilized value of 21°, which is in good agreement with the steady-state value (43° gives a reduced value of 32°).

As mentioned earlier, cavity expansion theory can also be used to estimate foundation bearing capacity. In a review of the approach, Dutt and Ingram[3] evaluated field data for platforms located in the Gulf of Suez and the Florida Gulf Coast and suggest that for calcareous sands, the mobilized friction angle is approximately 18° to 19°. They suggest a reduction of the form:

$$\O^* = \tan^{-1}(0.33 * \tan\O_p) \quad (6)$$

For a peak friction angle of 43° as obtained from the laboratory test data, this relation results in a mobilized value of 17°.

A comparison of the calculated bearing capacity using friction angles of 10°,20° and 30° is presented in Fig. 6[4]. For the expected maximum leg loads at the Río Caribe site (8 735 kips), the penetration will be of the order of 3 m (Ø= 30°) and 3.8 m (Ø = 10°). The difference between these two estimates is of little practical importance. However, there exists a discrepancy between the Rowan Odessa field data and the calculated Río Caribe data since the Rowan Odessa leg loads are much lower (6 000 kips), but the leg penetrations are similar to those calculated for Río Caribe. On the basis of the bearing capacity calculations, the Rowan penetrations should have been much smaller. The Rowan field data can be used as the basis for a back-calculation to:

- obtain the best-estimate of penetration at the Río Caribe site
- determine the actual mobilized friction angle at the Rowan site

On Fig. 6, the Rowan Odessa field data point for an average

SOIL PROFILE

The simplified soil profile for the site is shown in Fig. 1. A surface calcareous sand exists to a depth of 9.1 m, after which the complete profile consists essentially of stiff to hard clays. Based on initial liquefaction analyses, the sand layer is to be considered liquefiable to a depth of 15 ft (4.57 m) due to the expected ground accelerations at the site[2].

The variation of Atterberg Limits of the recovered samples is indicated in Fig. 2. The liquid (LL) and plastic (PL) limits are reasonably constant over the depth of the boring and give an average plasticity index (LL-PL) of around 40%.

Laboratory measurements of the submerged unit weight (γ') are also presented in Fig. 2, as is the smoothed trend used in the analyses performed.

STRENGTH PROFILE

The variation of the measured undrained shear strength in the cohesive soils is indicated in Fig. 3. The measured shear strengths have been obtained from several types of test and the scatter is that generally associated with offshore investigations. For the initial static analyses, the design profile indicated in Fig. 3 was used. For the subsequent dynamic and pseudo-dynamic analyses, the relationship from DSS and triaxial results (Eq. 1) has been used. Depending on the type of evaluation, this relationship is modified to take into account the effects of rate of loading and cyclic degradation.

$$(sur/\sigma_v')_{NC} = 0.276 \quad (1)$$

Fig. 4 presents the data from unconsolidated undrained (UU) tests in terms of the strain at 50% peak strength, as well as the remolded sensitive (s_{ur}) of the soils as determined from laboratory minivane and UU triaxial tests. The ratio of the peak (s_{up}) to remolded strength is used to determine the soil sensitivity (S_t):

$$S_t = (s_{up})/(s_{ur}) \quad (2)$$

The range of S_t values is in general agreement with the standard values for normal soils. Average values for the two types of test are 3.1 (UU triaxial) and 3.8 (minivane). The variation of the remolded strength with depth is shown in Fig. 3. The data suggest the following relationship.

$$(s_{ur}/\sigma_v') = 0.08 \quad (3)$$

STRESS HISTORY PROFILE

The stress history of the soils in terms of overconsolidation ratio (OCR) has been determined from both constant rate of strain (CRS) and incremental (IL) laboratory oedometer tests. The resulting OCR profile is shown in Fig. 4. OCR values have also been determined from the s_u/σ_v' ratio according to:

$$(s_{ur}/\sigma_v')_{OC}s = (s_{ur}/\sigma_v')_{NC} * (OCR)^{0.8} \quad (4)$$

The two sets of OCR values are in good agreement and suggest that the soils at the site are lightly overconsolidated at the surface (to a depth of about 30 m), becoming essentially normally consolidated with depth apart from a thin OC layer at a depth of about 60 m.

SHEAR WAVE VELOCITY PROFILE

The variation in measured shear wave velocity as determined from in situ and laboratory (Resonant Column) tests is shown in Fig. 5. The maximum shear modulus, G_o or G_{max}, has been calculated from the elastic relationship:

$$G_o = \rho * V_s^2 \quad (4)$$

where:

ρ is the bulk density of the soil (kg/m^3)

V_S is the shear wave velocity (m/s)

The majority of the data in Fig. 5 is taken from the laboratory measurements as problems were experienced in the execution, processing and interpretation of the in situ data.

CALCAREOUS SAND PARAMETERS

The basic soil parameters for the calcareous sands have been included in the profiles presented above. It was determined that the first 5 m of the sands would be considered as liquefied under the level of seismic ground accelerations expected at the site (this is discussed later in the paper).

In terms of static parameters, based on the initial interpretation of the laboratory tests, a drained peak friction penetration of 13 ft (4 m) corresponding to a leg load of 6 000 kips (27 000 kN) is plotted. Assuming that the development of bearing capacity with depth is similar to the trends suggested by the calculated values, the predicted penetration for a load of 40 000 kN can be obtained. The extrapolation on Fig. 6 suggests a total penetration of 5.8 m. This corresponds to a mobilized friction angle of approximately 9°.

Using the steady-state friction angle, $Ø_{ss}$, as a more reliable basis for correcting for compressibility, the data suggest the following relationship for estimating the mobilized friction angle in calcareous sands:

$$Ø^{*} = \tan^{-1}(0.44 * \tan Ø_{ss}) \quad (7)$$

If only the peak friction angle, $Ø_p$, is available, then the following correction should be applied:

$$Ø^{*} = \tan^{-1}(0.17 * \tan Ø_{p}) \quad (8)$$

This reduced friction angle was recommended as a design value for calculating both axial and lateral load capacities in the calcareous sands.

LOADING RATE CONSIDERATIONS

Soil strength and stiffness have been demonstrated to be visco-elastic in nature, that is, the actual magnitude is dependent on the rate of loading. Several static direct simple shear tests were performed to evaluate this dependency for the cohesive foundation soils. Strain rates of 1.3%, 200% and 41000% per hour were used in this series of tests. The shear strengths and secant shear moduli from each test were compared directly.

All samples tested showed a significant gain in shear strength and elastic modulus with increasing rate of strain[5]. The relationship between the increase in strength or modulus with increasing strain rate is approximately linear on a semilog plot and suggests an average increase of 15 % per log cycle increase in loading rate.

This result can be used in the cyclic and dynamic analyses required for evaluating the response of the foundation soils under differing loading conditions.

Cyclic DSS results were also used to determine the effect of number of cycles of loading on soil response. The degradation index, δ, where:

$$\delta = N^{-t} = G_n / G_1 \quad (9)$$

(N is the number of cycles, t is the degradation parameter and G is the secant shear modulus at N= 1 and n cycles)

was determinated from the change in modulus with number of cycles of fully-reversing harmonic strain. The t parameter obtained from the DDS results (t= 0,025 to 0,10) compared favorably with data from the Orinoco Clay and San Francisco Bay Mud[6] at cyclic axial strains in the range 0.1% to 1.4%. Again these results can be used for optimizing foundation design.

EVALUATION OF LIQUEFACTION POTENTIAL

The liquefaction potential of the surface calcareous sands was evaluated by means of both laboratory and field tests.

The information avaliable from the geotechnical investigation was such that the soil resistance to liquefaction could be evaluated by the following methods:

- SPT N-values obtained from measurements recorded during driving of the soil sampler

- evaluation of shear wave velocity measurements made in situ or in the laboratory using a direct correlation technique or by application of the cyclic strain method

- generation of pore pressure response during cyclic direct simple shear tests on recovered samples

Laboratory- based Evaluation

The results of the cyclic DSS tests on reconstituted samples of calcareous sand in terms of liquefaction resistance are displayed in Fig. 7 as a family of curves of average cyclic stress ratio versus number of cycles of loading at that particular stress ratio for which a specified percentage reduction in vertical effective pressure was obtained. (The reduction in vertical effective stress corresponds to an increase in pore pressure. The curve for a 100 % reduction in effective stress represents the condition of initial liquefaction.). Test were performed on a sample from the 20 ft penetration depth using nominal average stress ratios of 0.15, 0.3, 0.6 and 0.9. For a shear stress ratio of 0.3, approximately 20 to 30 cycles of loading would be required to produce initial liquefaction of the soil. This reduces to 14 cycles for a stress ratio of 0.5.

Results of response analyses performed for the site indicate an average stress ratio of 0.3 for the surface sand layer. For a magnitude 7.5 earthquake, Seed et al. suggest 15 to 20 equivalent uniform stress cycles. Hence, from a consideration of laboratory DSS results on remolded samples, it would appear that the probability of liquefaction to is quite high.

Evaluation Based on N Values

Undisturbed samples were obtained in the sand layers during the site investigation by driving a percussion sampler tube. A 175 lb (79.5 kg) sliding hammer dropping a distance of 5 ft (1.52 m) is used to drive-in the sampler, and the number of blows required to obtain 2 ft (0.60 m) penetration is recorded.

Based on experience (not avaliable in published form), Fugro McClelland applies an empirical correction to convert the obtained values into equivalent SPT values. Based on the corrected or equivalent N values at the Río Caribe site, Fugro McClelland conclude that for a surface acceleration of 0.25 g, liquefaction of the calcareous sand is very likely to depths of around 25 ft (7.6 m) [5]. However,the authors consider that the results are only indicative of the relative resistance to liquefaction throughout the sand layer and that a conclusive statement as to whether a particular layer will liquefy or not cannot be made with any certainty since:

- correction to N values is empirical and has not been technically validated.
- different conditions used in the test compared to recommended procedures suggested by Seed and Idriss for liquefaction assessment.
- use of empirical correlations, derived for silica sands, being applied to calcareous sands.

Evaluation Based on Shear Wave Velocity

In situ shear wave velocity measurements were made over four depth intervals using an upper source and downhole arrangement.

Of the four depth intervals, only two results were interpreted to provide estimates of shear wave velocity. For a 15 ft depth range starting from mudline, a shear wave velocity of 250 m/s was presented, whilst for the range 0 to 239 ft , 206 m/s was obtained.[5]

The value of 257 m/s for the first 15 ft (4.6 m) appears to be excessively high when compared to the N values reported for this depth range (N= 5 - 20) and would suggest that liquefaction of the calcareous sands would be very unlikely. The resonant column data (on reconstituded samples) also suggest slightly lower V_s values in this layer.

CONCLUDING COMMENTS

The paper presents a review of the geotechnical characterization performed for the Río Caribe offshore study area and presents some of the problems associated with the interpretation of in situ data obtained, especially in terms of liquefaction assessment. This interpretation is further complicated by the presence of calcareous sands at the surface.Fortunately the definition of design parameters in this thin surface layer was not critical to any aspect of the permanent foundation design.

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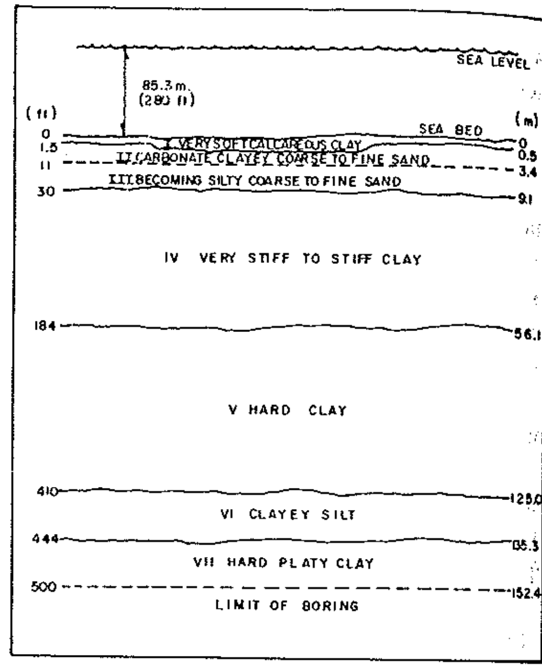

Fig. 1 Generalized soil profile for site B2

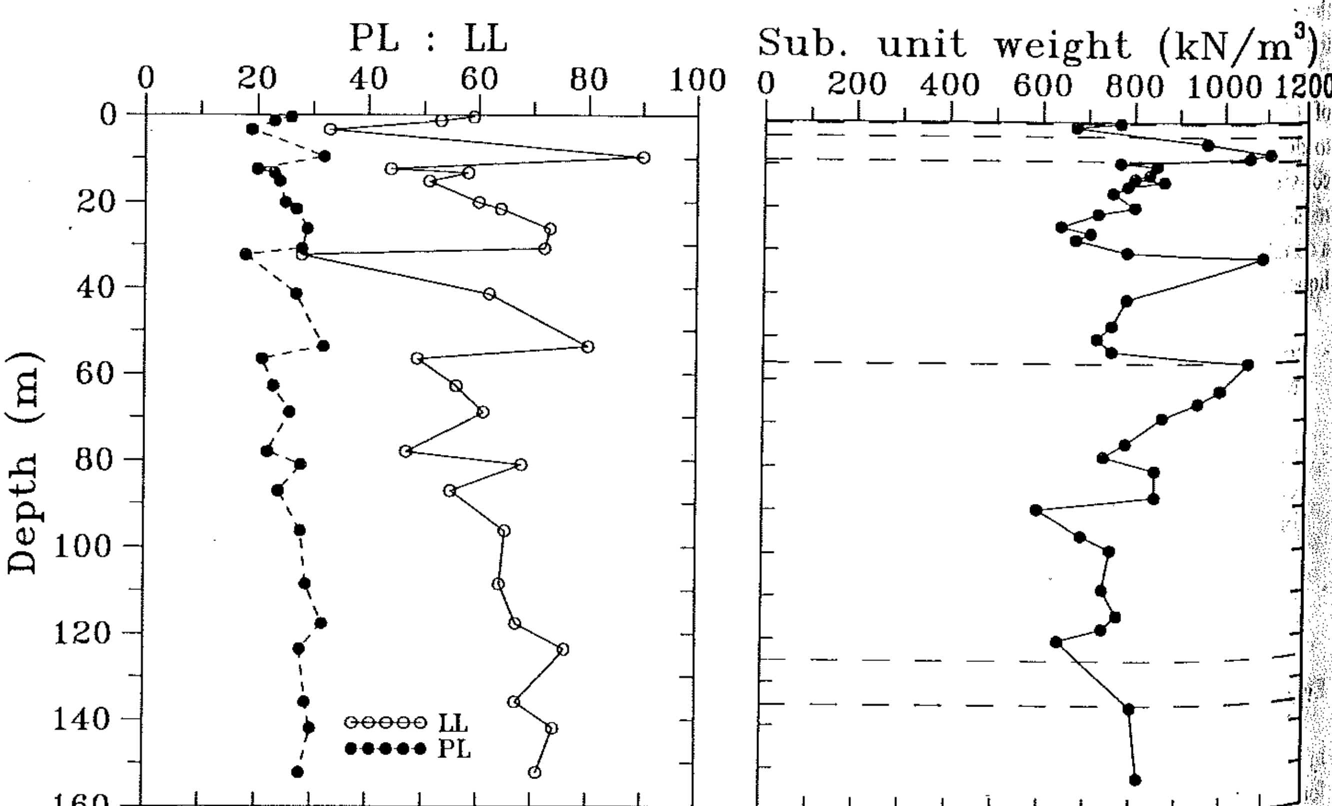

Fig. 2 Basic soil data from classification tests

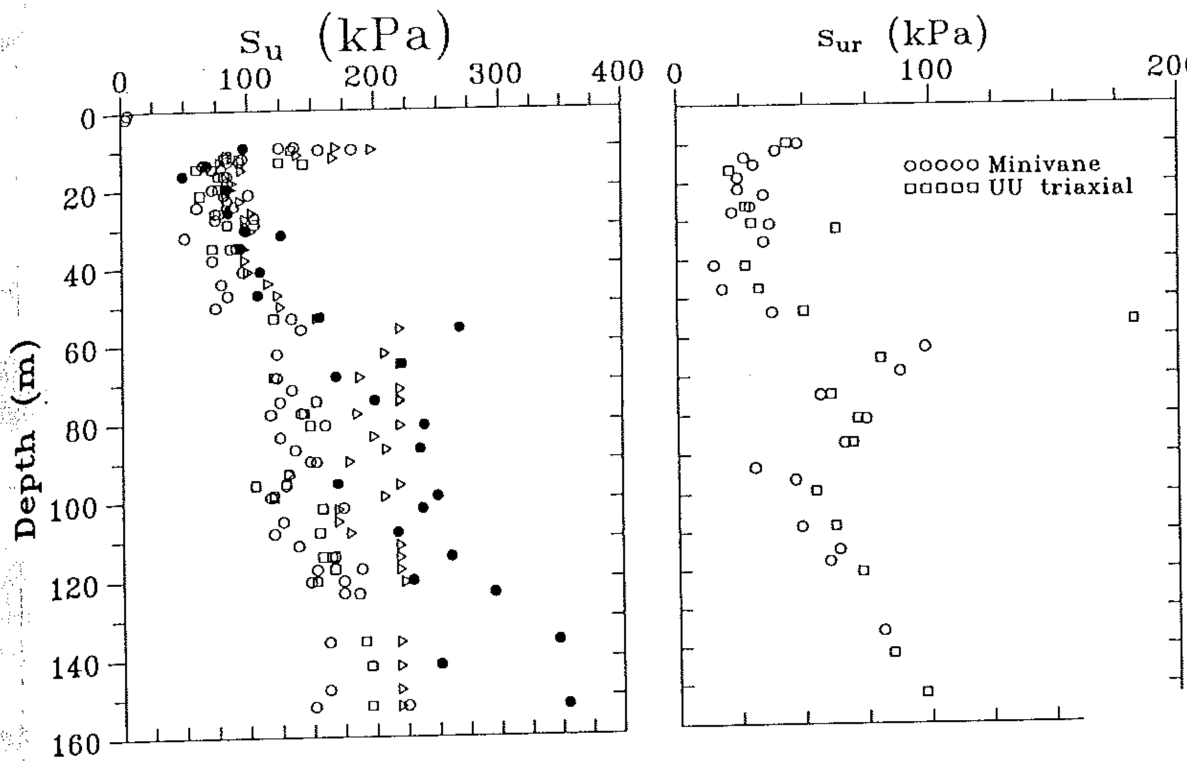

Fig. 3 Undrained peak and remolded shear strengths

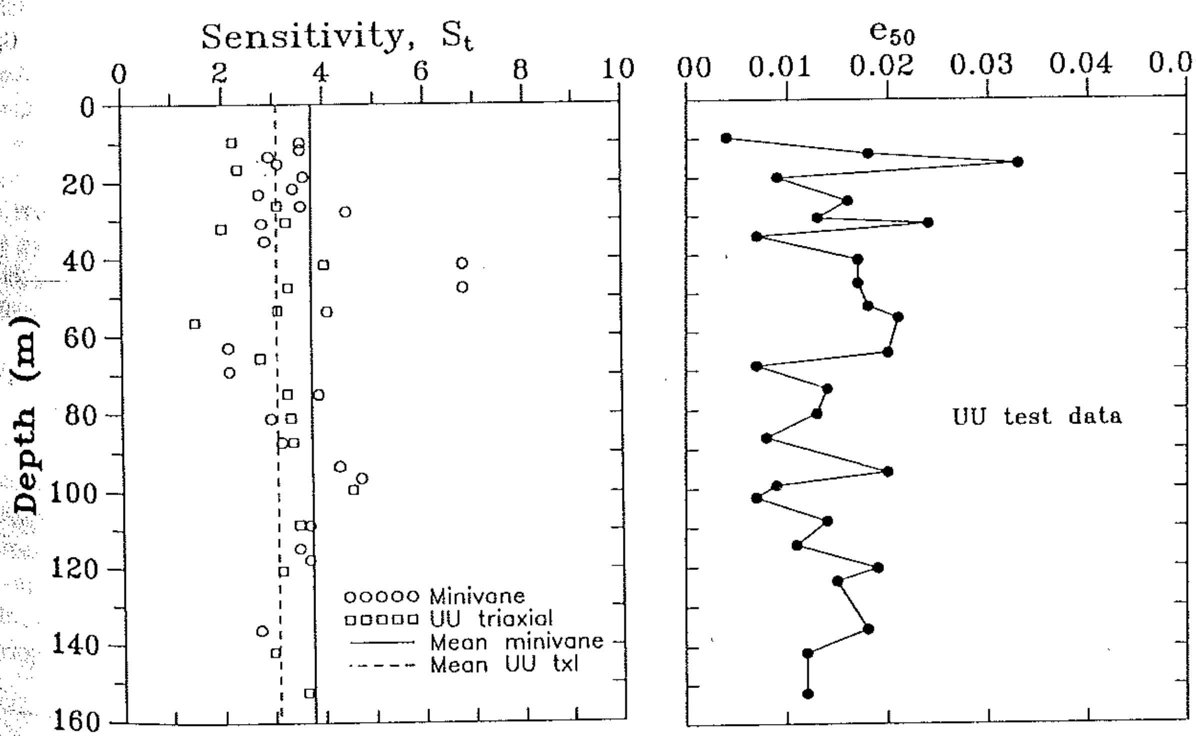

Fig. 4 Sensitivity and e_{50} values for clayey soils

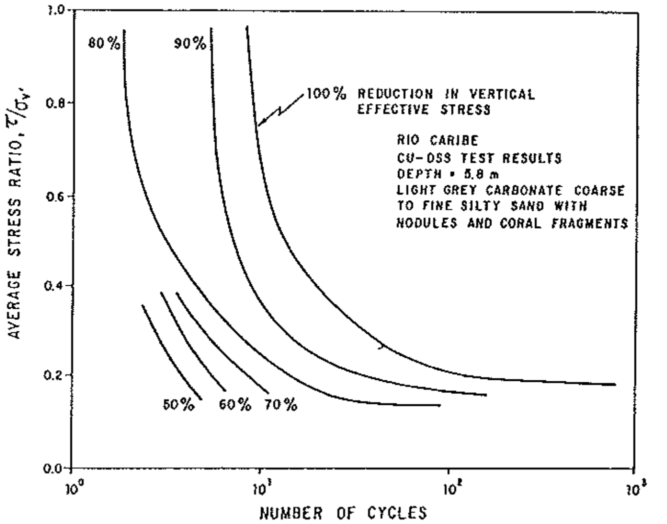

Fig. 7 Liquefaction resistance curves from cyclic DSS test on reconstituted calcareous sand

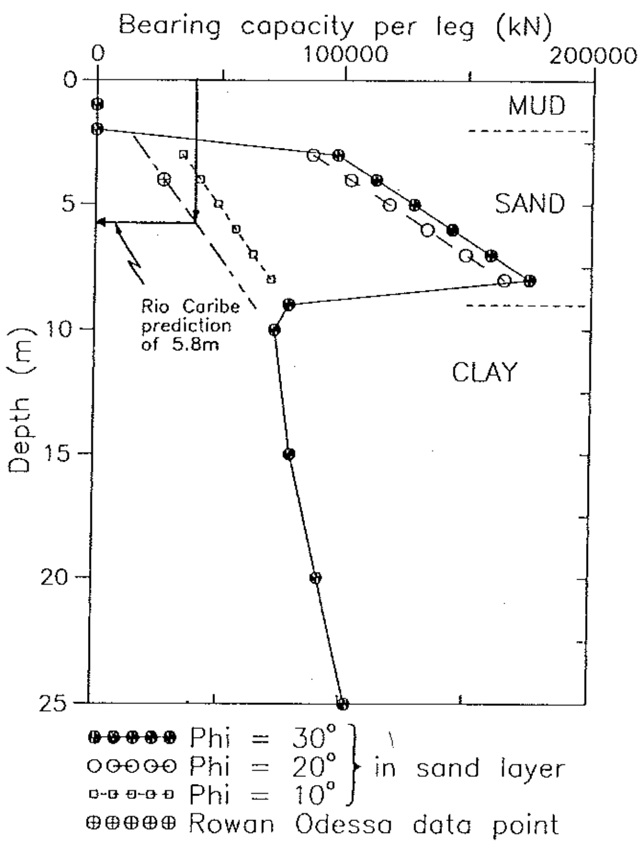

Fig. 6 Spudcean bearing capacity per leg for different friction angles

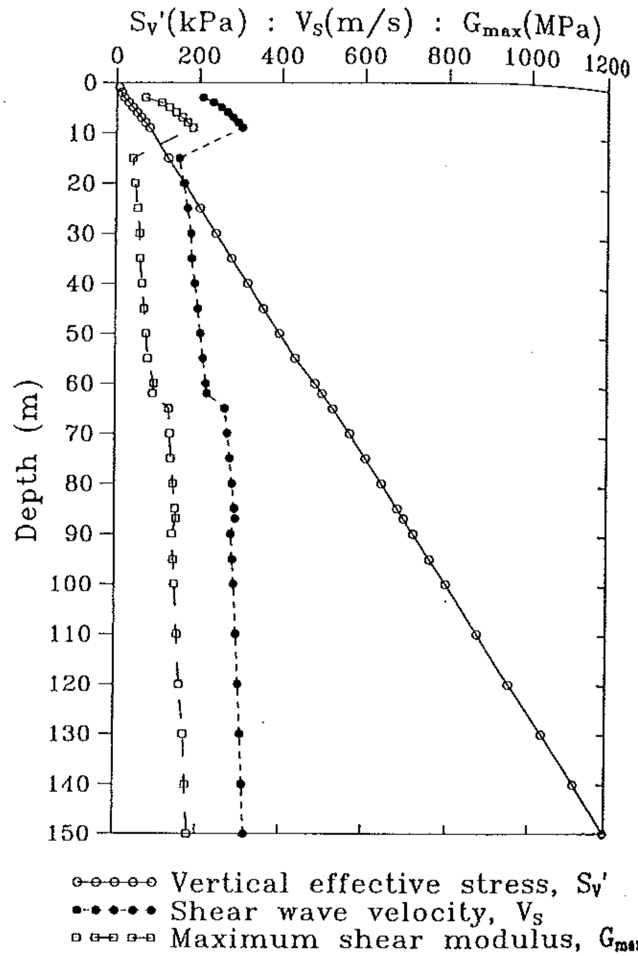

Fig. 5 V_s and G_{max} profile