OFFSHORE BERM BREAKWATER ON SOFT CLAY

BRISE-LAME DE BERME EN MER SUR ARGILE MOLLE

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SYNOPSIS: In Brazil a breakwater was constructed with unusual foundation conditions: 4 m of fine silty sand underlain by 8 m of soft marine organic clay under artesian pressure, 2.4 km offshore. The design studied by the authors has side slopes 1:3 and very wide berms in order to secure stability. A stage construction scheme was adopted to achieve necessary consolidation of the soft clay. Comprehensive instrumentation was installed to monitor pore pressures, horizontal displacements and settlements.

The backwards extrapolated virgin branches of consolidation tests were used for the determination of the preconsolidation stress of the aged clay. Undrained DSS tests showed that a strain of 5% led into creep failure. Recorded performance is related to calculated settlements and factors of safety. As a result of drainage, the clay in situ sustains larger shear strains, about 10%. The point of maximum strain moved from the bottom of the clay layer towards its middle while substantial creep took place.

INTRODUCTION

From 1988-92 a liquid bulk terminal was constructed near Aracajú, Sergipe, Brazil by Construtora Norberto Odebrecht S.A.(CNO) for Petrobrás. A breakwater, 600 m long, located 2.4 km offshore, protects the berths from wave attack. The rubble-mound breakwater was built directly onto the existing sea bed in 10 m of water.

After failure in 1989 of the initially constructed 100 m of breakwater the design was modified by the Brazilian geotechnical consultants Geoprojetos (GP) in cooperation with the Canadian hydraulic consultants Atria. The side slopes were reduced and the berms made much wider. A cross section of the redesigned breakwater appears in Fig. 1. A stage construction scheme was adopted to achieve the gradual consolidation necessary for stability: In the first stage the crown was at elev. +3.0 m, being raised in the second stage to +5.25 m.

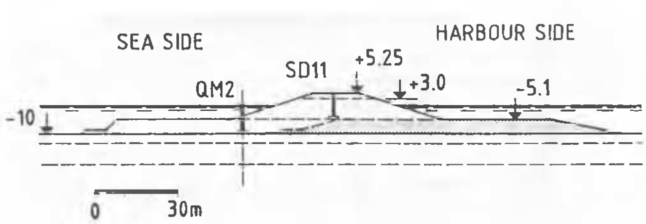

Fig. 1. Cross section of redesigned breakwater

SOIL CONDITIONS

A large number of boreholes indicate that the soil conditions are horizontally very uniform, as confirmed by piezocone tests (CPTU). The top layer is 4 m of fine silty sand, underlain by 7-8 m of very soft silty organic clay with marine shell fragments. The clay is believed to have been deposited in a protected deltaic lagoon 4000 to 7000 years B.P. The clay is underlain by a sequence of layers of fine silty sands, occasionally interbedded with silty clay layers.

Hydrogeological data show the existence of an artesian pressure of 28 kPa (with full tidal response) in the lower sand strata. The artesian water is almost fresh, the salt concentration increasing upwards through the clay.

SOIL PROPERTIES

Geotechnical Site Investigations

Following the initial failure in 1989, supplementary borings were carried out both in 1989 (in order to establish data for the redesign by GP) and in 1991 (for further studies of the project by COMAR, as well as by Professor C.C. Ladd, M.I.T.). These investigations included CPTU and field vane tests at three positions (B2-4) in 1989 and at one position (B5) in 1991. At B5 high quality samples were retrieved with a 100 mm Osterberg stationary piston sampler.

Laboratory Tests on Clay

A series of laboratory tests were carried out on clay samples from B5, ref. DGI 1991-92 and NGI 1991-92. The testing comprised: classification tests, oedometer tests, direct simple shear (DSS) and triaxial (CAU) tests, as well as dynamic DSS tests. The results are commented below.

Classification Tests

Results of classification tests appear from Fig. 2. The clay layer is seen to have a natural water content, w, close to the liquid limit, w_L.

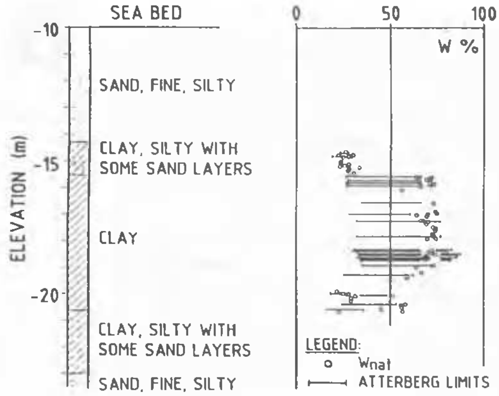

Fig. 2. Profile of geotechnical properties from borehole B5

For w = 68 %, γ_s = 27.2 kN/m^3, and γ_w = 10 kN/m^3 the unit weight γ of fully saturated clay is γ = 16.0 kN/m^3. Assuming that the artesian pressure of 28 kPa gives a constant gradient throughout the 8 m of clay, an effective unit weight of γ'' = 2.5 kN/m^3 is found. The in situ effective stress profile is plotted in Fig. 4.

Consolidation Tests

Even with the high quality sampler applied at B5, it proved difficult to obtain good test results on samples from the upper part of the clay, mainly due to sample disturbance caused by silt lenses. The best consolidation curve from the lower part is shown in Fig. 3.

Since the load was increased to as much as 4800 kPa, it was possible to use the strains along the virgin branch defined as end of primary consolidation (EOP), for a backwards extrapolation. By strict numerical extrapolation from the strains at stresses of σ_{vc}'= 600, 1200, 2400, 4800 kPa a value of σ_{vc}'= 68 kPa was found for zero strain, ϵ = 0, cf. Fig. 3. (At 150 kPa the laboratory curve falls below the virgin branch, presumably because of the extended duration of this load step (creep), and also at 300 kPa, presumably due to the unload/reload at 150 kPa.)

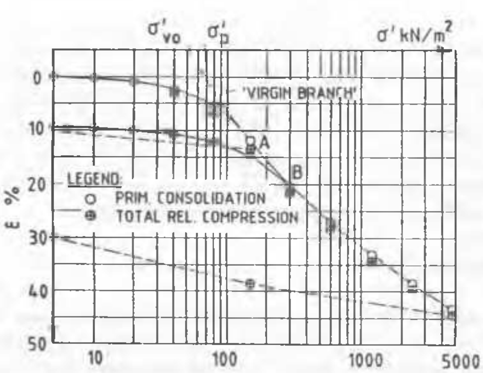

Fig. 3. Consolidation test No. DGI/1 (20 mm thick sample)

Apparently, the concave shape of the virgin branch shows a resistance against compression that increases faster than proportional to σ_{vc}'. Most of this increase is eliminated, however, if natural strain is considered.

It is reasonable to define the in situ preconsolidation stress σ_p' as the stress (68 kPa) of the virgin branch at the in situ void ratio (ϵ = 0). The "laboratory preconsolidation stress" is difficult to define because of the "disturbed" strain of 3% at the in situ stress of σ_{vo}' = 52.3 kPa (Table 1). It should be noted that the socalled "preconsolidation" is the result not of an additional load in nature but of aging (creep) under 52.3 kPa. A consequence of these considerations is that in situ loading above 52.3 kPa gives a very small compression before the (convex) load curve joins the virgin branch.

Values of σ_p' as here defined are listed in Table 1 and plotted in Fig. 4. For this clay it seems that a CRS test at a rate of 0.7%/h gives very nearly the same preconsolidation pressure as a standard oedometer test with incremental loading every 24 h.

Table 1. In Situ Preconsolidation Pressures from Consolidation Tests

Test No.	Elev.	σ_{vo}'	σ_p'	Test type	Strain rate	Sample quality
	m	kPa	kPa		%/hr	
NGI/1	-17.53	48.8	60	CRS	0.7	fair
NGI/2	-17.80	49.5	60	CRS	1.5	fair
NGI/3	-18.31	50.8	70	K_oOed		fair
NGI/4	-18.72	51.8	70	K_oOed		fair
DGI/1	-18.93	52.3	68	Oed		fair
DGI/2	-19.27	53.2	75?	Oed		(good)
DGI/3	-20.38	56.0	70	CRS	0.5	poor

CRS = Constant-rate-of-strain test
Oed = Standard 24 hr incremental loading
K_oOed = Oedometer with measurement of lateral stresses
Sample quality according to Tavenas et al. (1987)

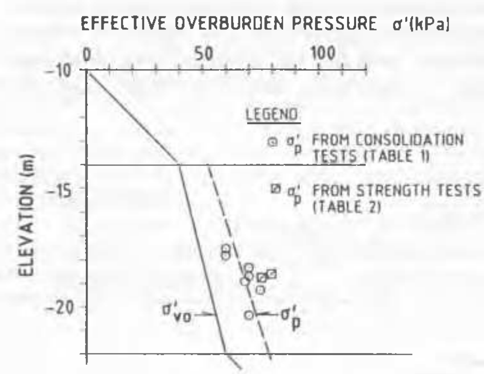

Fig. 4. Effective stress profile for borehole B5

Static Undrained Shear Strength

Results from three DSS and one CAU test are summarised in Table 2: The value of $(s_u/\sigma_{vc}')_{t=\infty}$ corresponding to an infinitely slow rate of shearing is estimated in accordance with Berre et al.(1973).

Table 2. DSS and CAU Shear Strength Tests

Test No.	Type	Elev.	σ_{vc}'	s_u	s_u/σ_{vc}'	$(s_u/\sigma_{vc}')_{t=\infty}$	Strain rate
		m	kPa	kPa			%/h
NGI/5	DSS	-18.67	52.2	16.2	0.310	0.298	0.25
NGI/6	DSS	-18.61	52.0	17.6	0.340	0.320	-
NGI/7	DSS	-18.37	118.3	28.6	0.242	0.232	0.25
DGI/4	CAU	-16.50	121.0	35.4	0.292	0.274	0.20
DGI/4	CAU	-16.50	121.0	38.8	0.321	0.286	2.0

For clays in the normally consolidated state in the direct shear zone, an undrained strength ratio, S(D) = s_u/σ_{vc}' = 0.23, is assumed (NGI/7).

The anisotropically consolidated triaxial sample DGI/4, where two strain rates were applied, gives an average of S = 0.280. The angle of friction is found to be 26.8°. Thus the normalized shear strength ratio along the failure line in the active zone is S(A) = 0.280 cos(26.8°) = 0.250. This is only 8% higher than the DSS strength ratio of 0.232. The strength ratio for the passive zone S(P) has not been determined. Based on general experience a value of S(P) = 0.18 is assumed.

If S = s_u/σ_{vc}' for normally consolidated clay is known, the preconsolidation pressure σ_p' can be found from the SHANSEP relation:

$$s_u/\sigma_{vc}' = S\,(\sigma_p'/\sigma_{vc}')^m \qquad (1)$$

when it is solved with respect to σ_p'. These calculated values of σ_p' are plotted in Fig. 4, using S = 0.23 and m = 0.8, ref. Ladd (1991). For test NGI/5 the value σ_p'= 71 kPa is found, confirming the values indicated in Table 1 for consolidation tests NGI/1-3.

Creep and Long Term Strength

A stress controlled DSS test was carried out in order to examine the influence of loading time on strain and pore pressure (NGI/6). A duration of minimum one day was used for each loading step. The resulting horizontal strains versus time from this test are plotted in Fig. 5 (final loading steps only).

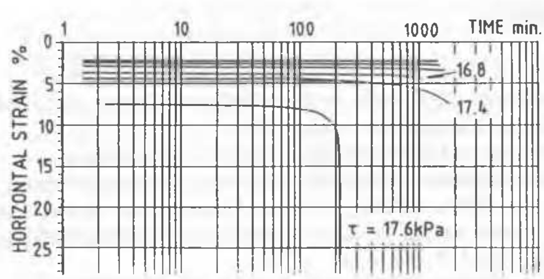

Fig. 5. Results from stress controlled DSS test NGI/6

At lower shear stresses a fairly constant logarithmic slope is seen below t = 1000 minutes. Failure due to creep took place at a shear stress of 17.6 kPa. It is probable that the previous step (17.4 kPa) with an initial shear strain of 4.5% would have given creep failure, if the duration had been longer. Judged from all available laboratory tests, it is tentatively concluded that the shear strength determined from static undrained shear tests may only sligthly overestimate, say, less than 10%, the undrained soil strength to be used for the evaluation of a creep initiated failure.

Coefficient of Consolidation

The coefficient of consolidation, c_v, has been estimated on the basis of settlement records. With due regard to the settlement inherent in the plastic flow and to secondary consolidation a value of 1.5 x 10^{-7} m^2/s has been found.

Cyclic Undrained Strength

Some cyclic DSS tests were made at NGI for the evaluation of pore pressure build-up and reduction of undrained shear strength when the breakwater is exposed to cyclic loading from waves. The results indicate that the Sergipe clay is subjected to less cyclic degradation than the Drammen clay so thoroughly investigated at NGI. The conclusion is that the wave action has negligible influence on the stability.

STABILITY

Strength Increase Due to Consolidation

The stability of the stage constructed breakwater is analysed in terms of total stresses, using the undrained shear strength determined from consolidated undrained tests.

There are two causes for strength gain during stage construction: (i) consolidation under shear stress, and (ii) consolidation under vertical load. An indication of the magnitude of the former can be found in Ladd (1991), Fig. 19, where there is an increase in strength of 15% for full shear consolidation of Atchafalaya clay if the initial factor of safety is 1.25.

With respect to consolidation under vertical load, the strength increase is 0.23 times the increase in effective stress. The increase in pore water pressure Δu_o (t = 0) resulting from the application of an additional load $\Delta p'$ is taken as $\Delta u_o = \Delta p'$ corresponding to undrained response. The resulting consolidation process is calculated from Terzaghi's one-dimensional theory, using a coefficient of consolidation c_v = 1.5x10^{-7} m^2/s for stress levels above σ_p'. Consolidation for loading up to σ_p' (Fig. 4) is assumed to appear instantaneously. Generally, this is slightly on the unsafe side, cf. Leroueil et al.(1990). In the present case, however, the error is negligible (Fig. 3).

Construction of Breakwater

The geotechnical berm (to elevation -5.1 m) on the sea side was constructed from December 1990 to April 1991. Thereafter the upper part of the breakwater was built to level +3.0 m from January 1991 to September 1991.

The final construction, to level +5.25 m, took place from May to October 1992, following advice from Professor C.C. Ladd.

Stability of Breakwater

The factor of safety (F.S.) was calculated for the situation in September 1991 with the breakwater constructed to level +3.0 m. The F.S. was determined by a limit equilibrium method using Janbu's

Simplified Method of Slices. Soil parameters for granular fill materials were taken from Lundgren et al. (1989). For the most critical failure surface (with the horizontal part of it at elevation -19 m in the soft clay) an F.S. = 1.4 was found.

REVIEW OF BREAKWATER PERFORMANCE

Site Instrumentation

The monitoring of the site was designed and analysed by GP. Settlement markers and inclinometers were installed in late 1990, following the construction of the geotechnical berm to elevation -5.1 m. The locations of these are shown in Fig.1.

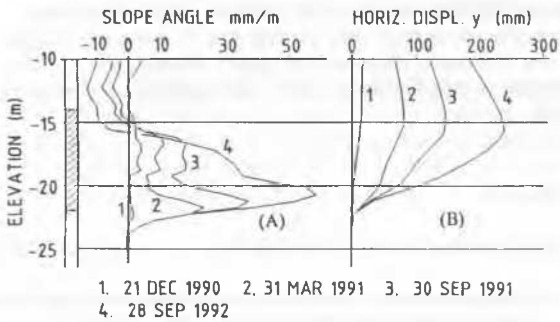

Fig. 6. Inclinometer readings versus depth

Measured slope angles versus depth in QM 2 are plotted in Fig. 6A and calculated horizontal displacements versus depth in Fig. 6B. The horizontal displacements are seen to increase rapidly upwards from the base of the clay (elevation -22 m) to reach a maximum about 2 m below the top of the clay. In general, the recorded shape of the horizontal displacements is similar to the form reported by Leroueil et al.(1990) with a maximum at a depth of about one fourth of the thickness of the layer. It is noteworthy, however, that the slope angles for a long time (curves 1-3) had two maxima, with a minimum at elevation -19 m.

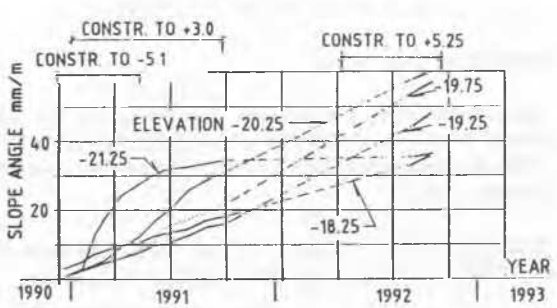

Fig. 7. Slope angles at various depths versus time

The recorded slope angles versus time at selected elevations appear from Fig. 7. It is seen that the location of the maximum slope angle shifted upwards with time. Initially the largest strain rate was found at elevation -21.25 m but later at -20.25 m. Eventually it is likely that the maximum strain will be found around elevation -20 m. The observed change in strain rates does not relate to the creep under actual shear stresses. Due to the longer drainage path the minimum strength gain could be expected to be found near the middle of the clay. From Fig. 7 the maximum final strain seems to be about 10%.

The relation between the settlement (s) at point SD 11 and maximum horizontal deformation (y_m) is indicated in Fig 8. It is seen that the ratio of Δy_m to Δs is approximately 0.30 during construction of the breakwater to elevation +3.0 m.

The volume, V_h, of the plastic "flow" associated with horizontal displacements is considerably less than the volume change, V_v, associated with settlements during construction ($V_h/V_v \approx 0.2$).

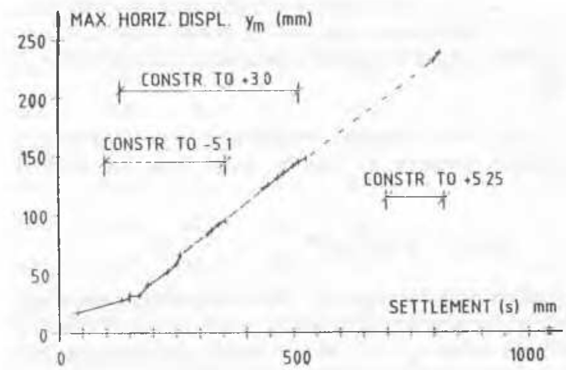

Fig. 8. Maximum horizontal displacement versus settlement

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