Stability of breakwaters on poor foundation

La stabilité des digues sur de mauvaises fondations

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SYNOPSIS If breakwaters, which are exposed to large wave forces, have to be placed on sites with weak foundation soils, it is customary to dredge the poor layers and replace them by sand. In connection with projects to offshore breakwaters in Brasil some new types of breakwaters have been developed. This paper discusses the geotechnical stability of various types of breakwaters and, in particular, the risk of liquefaction due to shaking by earthquakes or waves.

1 INTRODUCTION

In recent years there have been an increasing number of projects with breakwaters placed in locations with poor foundation conditions, cf. Lundgren and Lindhardt Jacobsen (1987). Such projects present a great challenge because of their combination of problems within construction, foundation and coastal engineering.

2 GEOTECHNICAL BACKGROUND

Breakwaters exposed to large waves and placed on poor foundation are so expensive that proper geotechnical investigations are of paramount importance. Deficiencies in the understanding of the geotechnical problems may result in unnecessary replacement of large volumes of soil.

2.1 Engineering Geology

Too often large harbours have been built without a thorough understanding of the engineering geology of the site. The investigations should normally start with the regional geology supplemented by a small number of borings in order to define the general stratigraphy of the area. The topographies of the various layers should be investigated by means of a seismic reflection survey.

One should always bear in mind that the sea level during the last glacial age was more than 100 m below the present one. This means that below the unconsolidated alluvial deposits there are always layers that have been exposed to dessication (stiff clays) or, e.g., to cementation by wave splash combined with heat (sandstones). An important part of the investigation is to locate buried river valleys and drainage channels from the glacial age. This knowledge is necessary to determine the optimum layout both of breakwaters (foundation) and of (easily) dredged areas.

2.2 Field Investigations

The detailed site investigations can be planned when the optimum breakwater alignment has been found on the basis of wave disturbance tests. The site investigation contract should be awarded to an experienced contractor and not necessarily to the lowest bidder. Otherwise the result may be an unsafe or an unnecessarily expensive design. The contract should not be on a lump sum basis but apply unit rates for the various items of work. The contractor should be carefully supervised by a qualified soils engineer.

The contractor should not be allowed to complete the whole job, leave the site and submit a report later on. On the contrary, upon completion of the first boring, the designer should start making estimates of the stability and settlements of a first tentative design. This will often result in the ordering of additional investigations while the contractor is still on the site.

As a minimum, each boring should comprise SPT, disturbed and undisturbed samples, and vane tests: (i) SPT-values should be taken right from the sea bed every 0.6 m until a depth of 2 m and, thereafter, every 1.2 m until N exceeds 50. (ii) Undisturbed samples should be taken at a maximum spacing of 2 m each time a new cohesive stratum is encountered. At least one boring with continuous coring should be made. (iii) Vane tests should be made every 2 m in all cohesive strata.

For fine-grained soils it can be of fundamental importance to determine the consolidation time for the weight of the breakwater, because it may be necessary to take into account the increased bearing capacity that accompanies the consolidation. Since all sediments have layers of coarser grains, the horizontal permeability can be several orders of magnitude larger than the vertical one. Consequently, it is important to make - with great care - in-situ permeability tests.

2.3 Normalized SPT-Values

It appears that SPTs have been performed in a manner that has been far from standardized. The result has been that the N-values found are not directly comparable. Reference is made to the European "Standard for the SPT test" and to Seed et al (1985), p. 1442, Appendix I: "Recommended SPT procedure for use in liquefaction correlations". Great care should be exercised in performing the tests in the recommended manner. In recent years the relationships between the various methods applied have been analysed thoroughly. These analyses are summarized below.

In 1975 H. Bolton Seed introduced the definition

$$N_1 = N\ C_N \qquad (1)$$

to correct for overburden pressure. Here

$$N_1 = \text{normalized N-value} \qquad (2)$$

i.e. the N-value that would be measured in the same stratum if the effective overburden pressure were 1 tsf (ton per square foot), where 1 ton = 2000 lbs and, hence,

$$1\ \text{tsf} = 0.975\ \text{kgf/cm}^2 = 95.7\ \text{kPa} \qquad (3)$$

In practice this is taken as 100 kPa. The coefficient C_N depends on the actual effective overburden pressure where N is measured.

Since the circumstances under which SPT tests are made, can vary in many different respects, it is, however, necessary to normalize the N-values recorded by altogether five correction factors:

$$N_1 = N\ C_{en}\ C_{rl}\ C_{sa}\ C_{bd}\ C_N \qquad (4)$$

Here

$$C_{en} = ER_r/60 \qquad (5)$$

is the factor that corrects the "rod energy ratio" to 60% as proposed by Seed et al (1985). The value (5) can be taken from Table 1, cf. Skempton (1986). The factors C_{rl} (for rod length), C_{sa} (for sampler) and C_{bd} (for borehole diameter) can be read from Table 2.

Table 1. Rod energy ratios for various hammers

	Hammer	Release	ER_r %	$ER_r/60$
Japan	Donut Donut	Tombi 2 turns of rope	78 65	1.3 1.1
China	Pilcon type Donut	Trip Manual	60 55	1.0 0.9
USA	Safety Donut	2 turns of rope 2 turns of rope	55 45	0.9 0.75
UK	Pilcon, Dando, old standard	Trip 2 turns of rope	60 50	1.0 0.8

Table 2. Corrections for rod length, sampler and borehole diameter

Corrections to measured N-values	
Rod length: > 10 m 6-10 m 4-6 m 3-4 m	1.0 0.95 0.85 0.75
Standard sampler US sampler without liners	1.0 1.2
Borehole diameter: 65-115 mm 150 mm 200 mm	1.0 1.05 1.15

There exists a good deal of uncertainty about the stress correction factor C_N. This is unfortunate because: (i) The examination of the liquefaction potential depends on N, and (ii) The increase of N with consolidation during construction may be utilized for developing the optimum design.

Laboratory tests on medium to coarse sands have shown a linear variation of N for the stress range 0 to 3 tsf, and field tests in fine sand are indicative of a linear variation for 0.5 to 2 tsf, cf. Skempton (1986). For linear variation the following relation applies

$$C_N = (A + 1)/(A + \sigma') \qquad (6)$$

where σ' is the effective overburden pressure (in kgf/cm^2 or tsf). For normally consolidated young sands Skempton gives examples of A in the range 0.9-1.3. Ageing tends to increase the range to 1.8-2.4 and overconsolidation to decrease it to 0.6-0.8.

There is a great need for the determination of A for silty and very silty sands. In breakwater projects with such soils there will always be much ageing (organic material may contribute to this) and, presumably, some apparent overconsolidation due to the compacting effect of the seepage forces set up by wave activity. For the upper 10 m below the sea bed it will be conservative to assume a large value of A. Without additional knowledge, a value of 2 or 3 may be used for A.

3. GEOTECHNICAL STABILITY PROBLEMS

3.1 Static Stability

For rubble mound breakwaters the critical stability condition occurs at LW and may be investigated by standard methods. Also the squeezing of a weak cohesive layer between more rigid strata and the breakwater may need to be analysed.

Correct evaluation of density and strength parameters are of special importance for breakwaters on weak soils. Breakwaters contain quarry run, rock fill and armour blocks. Below water level these materials are dumped directly, whereas they may be partly compacted, e.g. by dozer, above water. Parameter determination for such fills require large-scale tests. For a preliminary design, however, it is necessary to use existing data. Both the rock type and the quarry production and placement methods are relevant in this context. Also confining pressures, particle crushing and stress-strain properties may influence the field strength of rock fills. A summary of approximate rock fill strength and density parameters for use in preliminary design has been compiled from the literature, cf.e.g. Leps (1970), and is shown in Table 3. For sand fills the silt content is very important. Approximate parameters for clean sands are included in Table 3.

The required factor of safety for static loading should be in the range 1.4 to 1.5.

3.2 Semistatic Stability

The combination of LW with the design wave is designated the semistatic case. For the seaward slope the critical condition will be with the wave crest at the top of the slope and the wave trough outside the breakwater toe. The design wave may be e.g. the 100 year wave transformed into shallow water and corrected for reflection from the

Table 3. Parameters for rock fill and sand fill

Fill type		Rock fill		Sand fill	
Relative density	Effective stress kPa	Angle of friction	Unit weight kN/m^3	Angle of friction	Unit weight kN/m^3
Loose	30	44°	19.5	34°	19.5
	100	40°		32°	
	400	36°		30°	
Medium	30	50°	21	37°	20
	100	46°		35°	
	400	42°		33°	
Dense	30	56°	22	39°	20.5
	100	52°		38°	
	400	48°		37°	

breakwater. Since the wave breaks on the slope the weight of the water in the crest may be redistributed corresponding to an uprush covering the slope and penetrating the armour and underlying filter. Thus, for the semistatic case the superimposed wave loads are positive on the slope and negative below the trough. In a simplified approach no account is taken of the pore water pressure changes caused by the waves.

The required factor of safety in the semistatic stability analysis may be in the range 1.2 to 1.3.

A more accurate effective stress analysis involving pore water pressure changes due to wave loading is required where part of the fill or the foundation consist of loose sands which might liquefy under cyclic loading from wave forces. Sand fill pumped or dumped into water will be loose and may in certain cases have to be improved by compaction. Sands on the sea bed will normally be very dense due to rotating seepage forces from the wave load. Below a dense upper layer there may be loose, silty sands and soft clays deposited in calm water or in ancient river beds. The problems of wave induced pore pressure build-up and liquefaction are discussed in Sec. 3.4 below.

3.3 Liquefaction Potential for Earthquake Load

Fig. 1 shows a breakwater exposed to shaking under earthquakes as well as waves. The critical stratum in the layer of silty sand has to be examined for liquefaction.

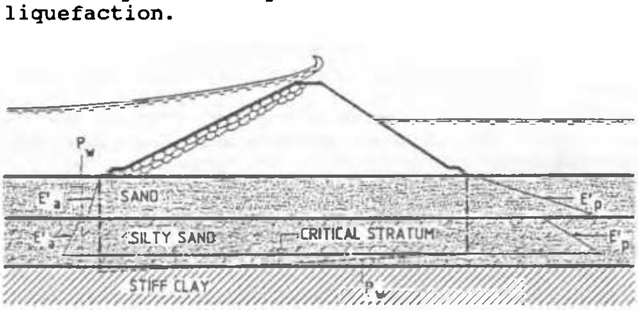

Fig. 1. Liquefaction of breakwater foundation

The mass m consisting of the breakwater plus the soil within the dash-dotted line is considered. It is exposed to an oscillating horizontal force giving a shear force S along the critical layer. In accordance with Seed et al (1985), the average shear is

$$S_{ave} = 0.65\ (m + m_h)\ a_{max}\ r \tag{7}$$

Here m_h is the added hydrodynamic mass from the water on both sides, a_{max} the maximum horizontal acceleration, and r is a stress reduction factor decreasing from 1 at the sea bed to 0.9 at a depth of 10 m below it.

The average permissible cyclic shear along the critical layer in Fig. 1 may be determined from

$$S_{ave,perm} = 0.011\ g\ m'\ N_1'\ R\ /\ F.S. \tag{8}$$

Here F.S. = factor of safety = 1.5, and the effective weight

$$g\ m' = g\ m - B \tag{9}$$

where B is the buoyancy. With good approximation, the tables given by Seed et al (1985) may be expressed by the following formulae. For an earthquake of Richter magnitude M the correction factor R in (8) is

$$R = 0.7 + 6/(NC + 5) \tag{10}$$

where

$$NC = \text{Number of cycles} = 0.12\ (M - 2.5)^3 \tag{11}$$

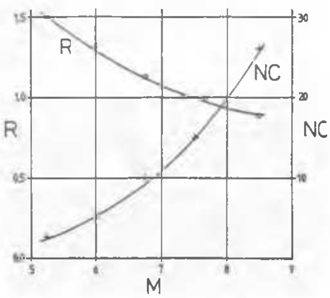

Fig. 2. Shear stress ratio and number of cycles

For M = 7.5, NC = 15 and R = 1. Thus, R is the ratio of the shear stresses that are allowed for earthquakes of magnitude M and 7.5, respectively. In Fig. 2 both R and NC are plotted as functions of M. The empirical values given by Seed et al (1985) are shown as individual points.

In (8) the normalized N-value has been adjusted for the percentage of fines (particles less than 0.06 mm) in the following manner:

$$\text{max } 5\%: \quad N_1' = N_1 \tag{12}$$

$$15\%: \quad N_1' = 1.1\ N_1 + 3 \tag{13}$$

$$35\%: \quad N_1' = 1.15\ N_1 + 6 \tag{14}$$

cf. Seed et al. (1985).

3.4 Liquefaction Potential Under Wave Loads

The shaking of a breakwater under wave loads is analogous to the shaking during earthquakes. In (7) the average shear is taken as one half of the average (over many waves) of the difference between the wave crest force (Fig.1) and the preceding wave trough force, as determined by model tests. In these forces are included the wave forces P_w in the soil. The 100 year storm may be taken as the design situation. This gives a risk of 1% the first year. Thereafter the soil resistance to liquefaction is increased, partly in connection with the reduction of the pore water pressure that was built up by preceding smaller storms, partly because of consolidation under the weight of the breakwater.

For wave shaking Eq. (8) becomes

$$S_{ave} = (0.011\ g\ m'\ N_1'\ R + E_p' - E_a')/F.S. \tag{15}$$

where the earth pressures E depend on the allowable motion. The wave force along the critical layer changes sign but has little influence. For many cycles NC Eq. (10) gives R = 0.7.

4. TYPES OF BREAKWATERS ON WEAK SOILS

It is expensive but quite common to replace soft layers by sand in situations where the bearing capacity is inadequate or the settlements excessive, cf. Lundgren and Lindhardt Jacobsen (1987).

In some cases interbedded sand layers may give sufficient drainage for consolidation to take place during the construction of the breakwater resulting in adequate bearing capacity. Then it may suffice to raise the crest of the breakwater.

If the construction time allows, it may be possible to use preconsolidation of the soft layers by building the breakwater in stages.

The risk of foundation failure from dead weight plus wave forces may be eliminated by using wide, stepped berms on one or both sides.

If the precautions mentioned above are insufficient, consideration may be given to improve the ground by sand compaction piles. Another possibility is to reinforce the fill at sea bed level with geogrid to improve the factor of safety against failure.

4.1 Piled Breakwaters

If the foundation conditions are very poor, gravity breakwaters are unsuitable. The wave forces may then be taken by a front wall of precast concrete elements supported on pile bents. Breakwaters with a vertical front are known. They have the disadvantage, however, that the vertical piles supporting the elements will be subjected to large, cyclic tensile forces. To reduce these forces, the structures sketched in Fig. 3 have been developed.

The front piles in Fig. 3a are raked as much as the construction equipment will permit. Because of the battered front, also the wave forces and, in particular, the shock forces are reduced. On the other hand, the battered front may also require that the breakwater height be increased, or a larger overtopping be accepted. In the composite structure Fig. 3b the forces in the piles are further reduced. The wide berm Fig. 3c induces breaking of the waves, reducing the wave forces. Fig. 3d indicates a further development, by which the submerged breakwater reduces the height of the waves that hit the wave screen to a minimum.

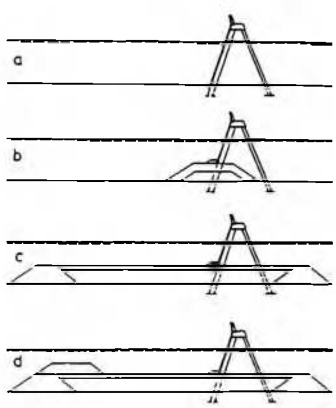

Fig. 3.
Piled breakwaters

Fig. 4 shows a variant of Fig. 3c, the stepped berm accounting for particularly weak ground.

5. CONCLUSIONS

Proper procedures for geotechnical investigations of breakwater sites are summarized.

Methods of stability analysis, incl. specifically the risk of liquefaction, are reviewed and developed.

Finally are given examples of types of breakwaters on weak ground and proposals for new types combining submerged rock fills with pile supported, inclined wave screens.

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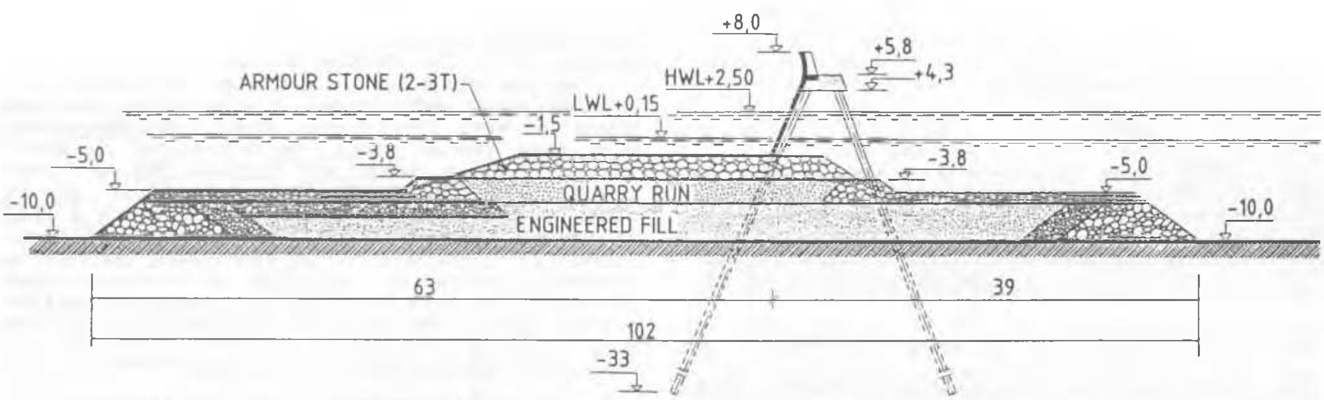

Fig. 4. Submerged breakwater with piled wave screen