LIQUEFACTION ANALYSIS FOR A BREAKWATER FOUNDATION

ANALYSE DE FLUIDIFICATION POUR FONDATION DE MOLE

R.K.M. Bhandari[1] **G.T. Vaidya**[2]

[1]Dy. Chief Consultant, [2]Manager
Engineers India Limited, New Delhi, India

SYNOPSIS: The studies presented in this paper pertain to assessment of liquefaction potential of replacement sand used as a foundation for a dynamically stable flexible rubble mound breakwater. The relevant data and boundary conditions as applicable to a project site situated at the West Coast of India have been used. The characteristic wave loading pattern and its sequence of occurrence has been evolved to represent more realistic wave action. A basis has been developed for the determination of most critical value of the number of waves in a wave group which produces critical state of wave induced shear stress in the foundation sand. The pore pressure history has been generated taking into account the dissipation and preloading effects due to moderate wave storms. It is shown that a partial liquefaction possibility exists in the upper 3 m of sand replacement due to first monsoonic wave action. The more severe Hurricane wave loading does not predict any liquefaction potential.

INTRODUCTION

For creation of a new harbour on the Western Coast of India, a dynamically stable flexible rubble mound breakwater has been planned. The configuration of harbour evolved to meet the functional design criteria envisaged a total breakwater length of about 6 km. As with any breakwater, the design of these dynamically stable breakwaters was governed by the characteristics of typical wave loading and the foundation soil. Additional constraints on design principles were put by the construction technique and phasing.

As part of the specialised investigation programe, a detailed geotechnical compaign was initiated to study specifically the design features of the breakwater foundation. It was revealed that a 9 m thick very soft to soft marine silty clay overlying sandy/lateritic clay followed by weathered/basaltic rock formations existed along the breakwater alignment. Such site conditions necessitated detailed analysis of various foundation alternatives to enable to select techno-commercially feasible foundation design for the breakwater. It was found from the stability analysis of the breakwater that the soft marine clay existing at and below the seabed needs to be dredged out to a depth of 9 m and replaced by suitable sand available within the harbour basin. A floating barge with controlled dumping capability was proposed for laying the sand fill in the foundation trench. Such a technique leads to rather low initial relative density of sand immediately after placement. Experience indicated that the initial relative density of sand placed in this manner may be of the order of 20%. Sand with so low relative density poses a threat to the stability of the breakwater due to its susceptibility to liquefaction under the severe wave climate encountered at the site.

Long breakwaters envisaged for the development of harbour required construction period to extend over some working seasons which essentially means that the construction activities would have to be phased out with respect to time and sequence of construction. These characteristic construction features would expose the breakwater foundation sand to monsoon wave action until the construction of breakwater mound resumes during the next fair weather season. This sequence of wave loading & construction leads to dissipation of the generated pore pressures with consequent densification of the seabed sand. The latter phenomenon is of considerable importance for assessment of the liquefaction potential. A theoretical basis which takes into account the features of pore pressure generation and dissipation has been used in the present studies for investigations into the liquefaction potential of the seabed sands.

CHARACTERISTICS OF WAVE LOADING

In the traditional method of liquefaction analysis, a series of irregular wave loading is converted into equivalent number of uniform cycles which cause an equal amount of pore-pressure build up. This approach is considered to be conservative as it does not include variations of the pore-pressure with regard to time history of irregular wave loading. In this paper, the irregular time history is looked at by considering a wave train comprising of certain no. of wave groups and each group followed by many smaller magnitude waves.

The primary wave climate prevailing at the site was classified into two main categories

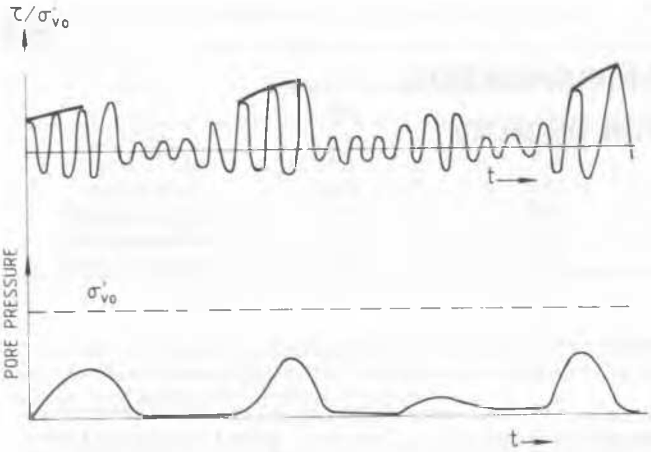

FIGURE 1 : ILLUSTRATION OF PRE-LOADING EFFECTS

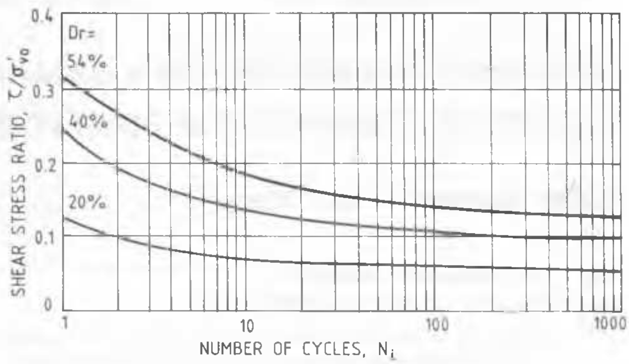

FIGURE 2 : SUSCEPTIBILITY TO PORE PRESSURE GENERATION

viz Monsoon and Hurricane. Both the wave types are highly irregular and follow a random sequence of occurrence. Detailed wave studies undertaken indicated that these waves often represented tendency to form groups. Each group could be characterized by a specific number of waves in it having some average height and period. Further, it was assumed that a trail of wave with high isolated peaks in the group is followed by a large number of small magnitude waves. This characteristic wave pattern would result in producing cyclic undulating shear stress pattern in the seabed which is typically represented in Figure 1. A detailed analysis of wave climate led to the basic wave parameters which are shown in Table 1.

TABLE 1 : BASIC WAVE PARAMETERS

TYPE OF WAVE	SIGNIFICANT WAVE HEIGHT, Hs(M)	WAVE PERIOD,TP (SEC)
Monsoon	3.1	14 Sec.
Hurricane	5.3	12 Sec.

It is considered here that each increment in the number of loading cycles results in proportionate increase in the cyclic generated shear stress until its critical value is reached. In order to select conditions producing highest pore pressures, several groups of waves each containing different number of waves were examined. Accordingly, a series of wave groups containing 3, 10 and 30 number of waves were studied to obtain the critical shear stress. The average equivalent wave height of the groups was derived using Longuit - Higgins theory. The computed wave heights are presented in Table 2 which also takes into consideration the long term variations in wave heights for both the wave types.

TABLE 2- EQUIVALENT WAVE HEIGHT OF A GROUP

TYPE OF WAVE	EQUIVALENT WAVE HEIGHT, H(N)*		
	N=3	N=10	N=30
Monsoon	6.0 m	4.8 m	3.22 m
Hurricane	9.4 m (depth limited)	8.2 m	5.51 m

*H(N) is the average height of 'N' successive largest waves of a group.

Critical value of N relevant for liquefaction is discussed later in the paper.

THEORETICAL CONSIDERATIONS

Liquefaction potential of seabed is determined by conducting cyclic triaxial laboratory testing on the undisturbed field samples. However, for the wave loading conditions which last for longer duration, the dissipation of generated pore pressure and its build up during subsequent wave cycles becomes more important. These phenomenon would lead to successive changes in the properties of sand viz - porosity and relative density. In the present studies, although no laboratory tests with pore pressure dissipation were carried out, the improvement in relative density of sand was utilised in the analysis. The model used to evolve a relationship between shear stress ratio τ/σ'_{vo} and number of cycles to liquefaction Nl was based on that given by Barends and Calle (1985), refer Figure 2.

For the representative wave loading considered here, the generation of pore pressure will start with the onset of Monsoon when an average significant wave height of about 2m is reached. As the wave group grows further in intentsity, the pore pressure will also grow and will dissipate when the storm receds and smaller magnitude waves prevail. This would lead to pre loading which is charaterised by densification of the seabed sand and is related to various factors such as the duration of the storm, rate of pore pressure generation and the drainage capacity factor θ given by $\sqrt{2ct}$.
(for symbols, refer below).

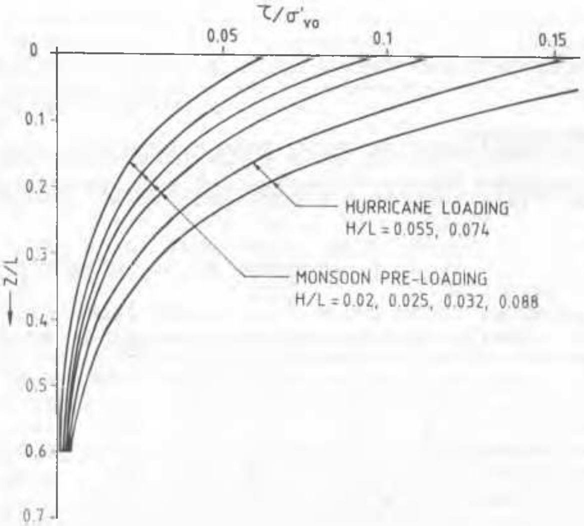

FIGURE 3 : SHEAR STRESS RATIO FOR MONSOON AND HURRICANE CONDITION

The hydrodynamic period required to achieve consolidation is considered to be proportional to D^2/C_v with the assumption that the pore water drainage takes place at the seabed. The cyclic shear stress, τ/σ_{vo} induced in the seabed is computed assuming that the soil is homogeneous, porous elastic half space. The shear stress ratio τ/σ_{vo} is given by (Barends and Calle ,1985);

$$\tau/\sigma'_{vo} = \xi/\gamma' . \exp(-\lambda . z) \qquad (1)$$

where

ξ = wave pressure steepness $\hat{P}\lambda$, $\hat{p}$ = Amplitude of wave induced pressure at seabed, λ = $2\pi/L$, L = wave length, γ' = submerged volumetric weight of soil, z = depth below seabed.

The rate of pore pressure generation, ψ_o , due to reversal of shear stress is obtained using the model of Barends and Calle (1985) which is given as

$$\psi_o = b . f_n . \gamma \psi' \sigma'_{vo} \left[(\xi - \xi_\infty)/\gamma' \right]^a \qquad (2)$$

where

'a' and 'b' are dependent on porosity of sand, r has been assigned value of 1 and 0.64, fn - loading frequency, ψ' = 2.1, and ξ_∞ - threshold value of ξ below which no pore pressure generation starts.

The attenuation of ψ_o with depth is obtained by using the following equation

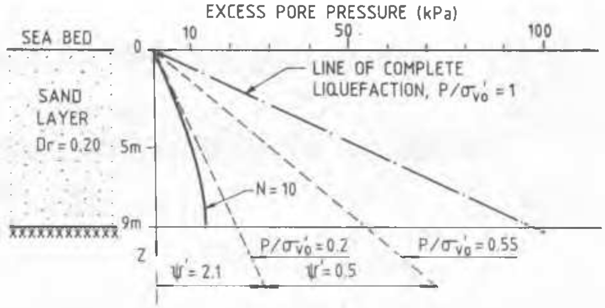

FIGURE 4 : PORE PRESSURE BY MONSOON WAVES, Hs=2m

$$\psi_o = \psi_{o} \exp(-a \lambda . z) \qquad (3)$$

The above formulation for ψ has been utilised in the determination of excess pore pressure during the action of various wave groups. In the computation of excess pore pressure it is assumed that the rate of generation is independent of its dissipation. However, the successive changes occurring in the properties of sand viz. porosity, relative density due to preloading have been taken into account in the analysis. The relative change in the relative density of seabed occurring at the end of each wave group action has been modelled as :

$$\Delta Dr \Big|_{Z=-D} = 1/0.14 \int M_v \psi_o \exp(-a \lambda D)\, dt \qquad (4)$$

where, Mv is the coefficient of compressibility, ΔD_r is the relative change in the relative density.

Excess pore pressure is then computed using the following relationship (Barends and Calle, 1985):

$$P = 2t\psi_o/(1-(a\lambda\theta)^2)\Big[\exp(-a\lambda z) - \big\{\cosh(D-Z/\theta) - a\lambda\theta . \exp(-a\lambda D) . \sinh Z/\theta\big\} 1/\cosh D/\theta\Big] \qquad (5)$$

where, t - storm duration, P-excess pore pressure.

LIQUEFACTION ANALYSIS AND DISCUSSION OF RESULTS

The amplitude of wave induced shear stress ratio at seabed and its attenuation with depth has been computed for various wave steepnesses for both the Monsoon and Hurricane wave conditions. These results are plotted in Figure.3. Using the maximum value of shear stress ratio, and the in-situ relative density of sand, the number of cycles required to cause liquefaction has been obtained for each group of wave from Figure 2. However, before any liquefaction susceptbility study is made, the influence of number of wave cycles on the shear stress has to be investigated. For this purpose, the increase in shear stress ratio as a result of increase in the number of cycles

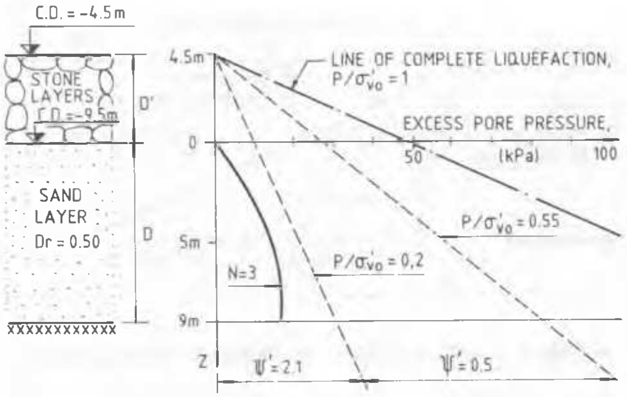

FIGURE 5 : PORE PRESSURE BY LARGEST WAVE, Hs=5.3m

of loading was calculated using Figure 2. This increase assessed in terms of increase of equivalent loads is as given below :

INCREASE IN NO. OF CYCLES,N	EQUIVALENT LOAD INCREASE
1-3	1.25
3-10	0.98
10-30	0.75

From these data, it is observed that no. of load cycles which induces highest pore pressure can be anywhere between 3-10. The equivalent wave heights corresponding to these numbers and the no. of cycles to liquefaction are shown in Table 3.

Excess Pore Pressure Due to Monsoon

As breakwater construction was not expected to commence before the end of monsoon, the pore pressures have been calculated for the construction phase when only the sand filling was in position up to the seabed. The various input parameters required for the computation are presented in Table 3. . These include equivalent wave height H, number of waves in the group N, relative density Dr, porosity n, permeability K (assumed to be isotropic) and coefficient of consolidation Cv. The final results obtained are presented in Table 3, while figure 4 depicts one typical result for the case of first monsoon wave group. It is inferred from this figure that the excess pore pressure generated is quite high and the sand fill upto a depth of 3 m is likely to liquefy, at least partially. The increase in relative density due to the action of successive wave groups is also shown in Table 3. It is seen that to achieve a relative density of 40%, a minimum of four number of wave groups from a single monsoon are required.

TABLE 3 - INPUT PARAMETERS AND RESULTS OF PORE-PRESSURE CALCULATIONS

Wave group	Monsoon				Hurricane	
	1	2	3	4	5	6
Parameters						
D' (m)	0	0	0	0	-4.5	-4.5m
D (m)	9	9	9	9	9	9
Hs (m)	2.0	2.6	2.6	3.1	3.6	5.3
N	10	10	3	3	3	3
Dr	0.20	0.30	0.30	0.40	0.40	0.50
n	0.362	0.348	0.348	0.334	0.334	0.320
k (m/s)	---		0.0006			-----
1/Mv (Kpa)	17000	23000	23000	29000	48000	57000
σ'_{vo} (Kpa)			50			
H (m)	3.1	4.0	5.0	6.0	7.0	9.4
γ'	10.14	10.37	10.37	10.59	10.59	10.88
Nl	2.4	193	16	38	4.4	3.6
ψ_c (Kpa/s)	0.31	0.038	0.48	0.20	1.26	1.57
Cv (m2/s)	1.06	1.43	1.43	1.80	3.0	3.6
P(D) (Kpa)	11.0	1.0	10.8	3.8	14.6	15.7
ΔDr(D)	0.008	0.001	0.003	0.001	0.003	0.003

Excess Pore Pressure Due to Hurricane

The toe of the breakwater is known for its susceptibility to failure due to liquefaction. Therefore, the excess pore pressures have been computed for the critical situation when the toe is laid at -4.5m C.D. The results are shown in column 5 & 6 of Table 3 from which it can be seen that the build up of excess pore pressure is rather limited. Figure 5 shows that the excess pore pressures developed due to hrricane are well within the confining pressure at all depths and therefore the possibility of liquefaction due to hurricane was discounted. Based on these findings, it was decided to use well graded coarse sand in the upper 3.0m of the fill to avoid any possibility of partial liquefaction due to monsoon waves.

CONCLUDING REMARKS

A case study has been presented to assess the liquefaction potential of a sand replacement underneath the breakwater. The procedure adopted in the analysis takes congnisance of breakwater construction sequencee, the charateristics of wave loading, dissipation of pore-poressure and the pre-loading effects for the entire range of monsoon and hurricane wave groups. Due to low initial relative density of 20% a partial liquefaction possibility was observed for the first group of monsoon waves. However, during hurricane loading, restrained generation of pore pressure due to pre-loading led to discounting of liquefaction. It is concluded that for safeguarding the stability of the breakwater, a remedial measure consisting of replacement of upper 3m of sand fill by clean coarse sand must be undertaken.

REFERENCE

Barends F.B.J & Calle E.O.F (1985). A Method to Evaluate the Geotechnical Stability of Offshore Structures Founded on Loosely Packed Seabed Sand in a Wave Loading Environment. Proc. 4th Int. Conf. on BOSS, Delft, The Netherlands.