A simplified procedure to assess the dynamic stability of a caisson breakwater

Une procédure simplifiée pour évaluer la stabilité dynamique d'une digue en caissons

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ABSTRACT: The paper describes a simplified method of analysis used to evaluate the stability of a caisson breakwater to sea wave actions. An intensive laboratory program was performed in order to evaluate the static and dynamic characteristics of the foundation soil. Anisotropic and isotropic consolidated cyclic triaxial tests and cyclic simple shear tests were used to define the cyclic interaction diagram for the foundation soil. The possibility of foundation cyclic mobility due to wave loading and their effect on the breakwater stability was examined combining the cyclic interaction diagram with the results of finite element analysis. The potential reduction in soil strength is then incorporated into a conventional stability analysis. The procedure is illustrated by a specific application to a caisson breakwater that is part of the extension works of the Barcelona Harbour.

RÉSUMÉ : L'article décrit une méthode simplifiée pour évaluer la stabilité d'une digue verticale sous l'action de la houle. Les caractéristiques statiques et dynamiques de la fondation ont été évaluées à l'aide d'un programme intensif de tests en laboratoire, qui inclut des essais triaxiaux cycliques isotrope et anisotrope et des essais de cisaillement simple cycliques dans le but d'établir le diagramme d'interaction cyclique du sol. La possibilité d'une mobilité cyclique de la fondation sous l'action de la houle et son effet sur la stabilité de la digue ont été examinés en combinant le diagramme d'interaction cyclique ainsi obtenu avec une analyse numérique par Éléments Finis. La réduction potentielle de la résistance du sol est ensuite incorporée dans une analyse de stabilité conventionnelle. La procédure est illustrée par une application spécifique à une digue en caissons qui fait partie des travaux d'extension du port de Barcelone.

KEYWORDS: cyclic tests, interaction diagrams, liquefaction, caisson breakwater, wave loading, stability.

1 INTRODUCTION

Two new breakwaters and a large container area, immediate to a new quay, are the main development works of the ongoing extension of Barcelona harbour. A plan view of the new breakwaters and quays is shown in Figure 1.

Breakwaters have a total length of 6.8 km. The East breakwater is of a rubble mound type whereas the South breakwater involves two different types: rubble mound and vertical caissons. This paper refers to the caissons section that has a total length of 1.7 km constructed in water depths that range from 20m to 25m. Most of the foundation soil immediately under the breakwaters consists of weak sediments of clayey silts and silty clays belonging to the pro-deltaic deposits of the Llobregat River.

The paper describes summarily the main geotechnical features of the foundation ground with special attention given to undrained strength parameters. The bases for the static design of the breakwater are then briefly presented. Finally, a description of the cyclic resistance of the foundation soil is described in terms of an interaction diagram; this information is then used in a simplified assessment of the stability of the breakwater under storm conditions incorporating the potential strength reduction due to cyclic loading.

2 SOIL PROFILE CHARACTERISITCS

A representative soil profile at the location of breakwaters is shown in Figure 2. It consists of: i) upper silts and clays, brown and grey in colour, although dark colours occasionally appear when organic matter content increases. The thickness of this deposit underneath the breakwaters is about 50 m. Sandy and silty sand inter-stratifications, were often found, specially in the upper levels of the layer. ii) an intermediate layer of gravels and sands, whose thickness is about 7 m; some silt partings were also detected. iii) a lower level of clays whose identification properties are similar to the upper clay unit, although it is a denser soil. The maximum thickness of this layer is 14 m. iv) a lower layer of gravels and sands; it includes several clays and sands stratifications.

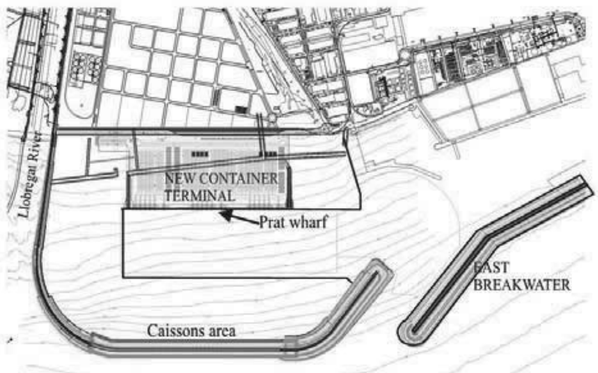

Figure 1. Plan view of the new breakwaters and new container areas of the Barcelona harbour. The location of the caisson breakwater is indicated.

Closer to the coast line, an upper deltaic sand deposit of increasing thickness, laid on top of the upper stratum of soft silty clays, appears. As it would be expected from a deltaic environment the transitions between this sand deposit and the upper clays are neither sharp nor regular. This sand deposit is 15 m thick at the shore line but it practically disappears at the breakwater location and it is not considered further in this paper.

Some of the geotechnical indices and properties obtained during the site investigations are shown in Figure 3. The fine grained materials classify mainly as CL (low plasticity clays) and ML (low plasticity silts). Water content commonly exceeds the liquid limit in the upper part of the soft silty clay unit, but at lower levels it is close to the plastic limit, an indication of the self weight consolidation of the sediments. Void ratios range between 0.8 and 1.0 in the upper clay stratum. Dry densities vary from 1.2 Mg/m^3 at the upper clay levels to 1.8 Mg/m^3 at deeper locations.

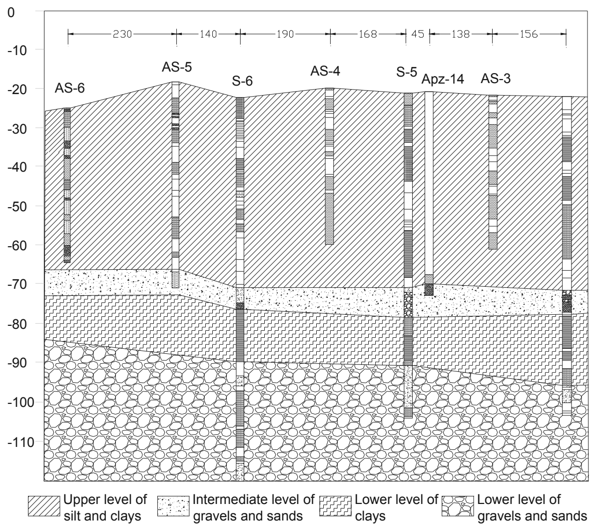

Figure 2. Soil profile under the caisson breakwater

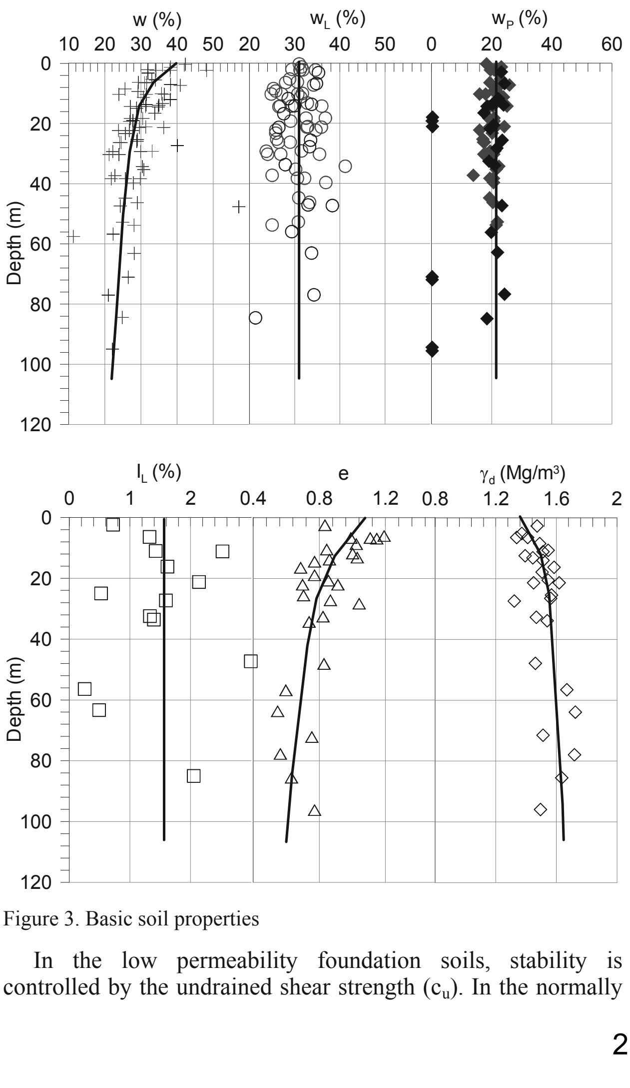

Figure 3. Basic soil properties

In the low permeability foundation soils, stability is controlled by the undrained shear strength (c_u). In the normally consolidated range, this parameter is largely proportional to the consolidation effective vertical stress. Undrained shear strength has been examined by means of laboratory and in situ tests.

Unconfined compression tests of clay samples provided a value of c_u=0.215σ'$_v$. However, sample disturbance and suction loss may lead to an underestimation of the real value (e.g. Tsuchida, 2000). Simple shear tests performed by NGI provided a value of c_u=0.25σ'$_v$. quite consistent with the results of CPTU tests. Anisotropically consolidated triaxial tests (compression and extension) yielded a range of c_u=0.21 – 0.33 σ'$_v$, the larger values associated with compression tests. A summary of results obtained is presented in Figure 4. The unusually large values of undrained strength obtained in some vane tests were probably due the occasional presence of sand lenses or laminations.

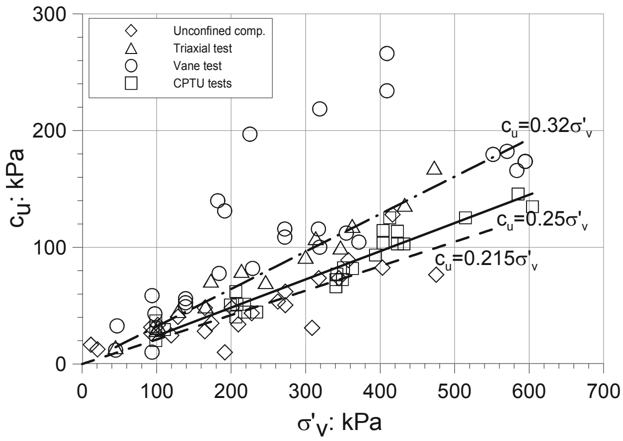

Figure 4. Undrained shear strength. Summary of results

It was also found that specimens sheared under normal effective stresses reproducing in situ stress conditions showed somewhat higher strength ratios than specimens consolidated to higher effective stress values. This is an indication of some modest overconsolidation/structure effects due to natural creep or aging phenomena. However, the additional stresses applied by the caissons and fills will take the soil in situ to a normally consolidated state. Therefore, a conservative attitude is favoured for the selection of the undrained stress ratio. The static design of the breakwater was eventually performed using a value of c_u=0.25σ'$_v$.

3 BREAKWATER DESIGN

The conventional breakwater design was performed using finite element analysis as the most efficient method to consider automatically the variation of undrained shear strength throughout all stages of construction. The following phases were considered: i) dredging and bench construction on the new soil surface, ii) caisson placement and filling, iii) construction of the superstructure, and iv) backfill behind the caissons to create a new quay zone. Although all potential limit states were considered, it should be pointed out that the use of finite element analysis readily identifies the most critical failure mechanism at every stage of the analysis. It should also be noted that the gain in undrained shear strength during each one of the construction phases was a critical feature with respect to the stability of the subsequent construction phase.

The wave and uplift forces due to storm loading in the different phases of construction are listed in Table 1. They were derived from physical model tests using the specific breakwater design. Wave forces depend on two factors: the height of the superstructure that provides the surface on which the wave impact acts and the wave height that in turn depends on the intensity of the storm. It can be observed that the wave height (and hence the storm intensity) is lower in Phase II. This is due to the temporary character of this Phase that makes it less likely that an extremely large storm will occur during that limited period. Probability analysis based on available time series provides the design storm to be used in each particular stage.

For each construction stage, a variety of factors of safety were used to assess the degree of stability of the breakwater affecting either loads or soil strength parameters. In the former case, wave caisson weight and storm wave loads were considered both jointly and separately. The values of safety factors were assessed in relation with the perceived uncertainty of the parameters involved. Thus, a higher factor of safety was demanded when only the wave action was considered due to the much larger uncertainty of the load magnitude associated with the storm. In fact, uncertainty affects both storm intensity and the actual effect on the caisson. The final design of the breakwater is depicted in Figure 5. Note the wide rockfill bench required for stability. An example of the failure mechanism in a particular instance of the analysis is shown in Figure 6.

Table 1. Wave and uplift forces acting on the caissons breakwater at different phases of construction

Phase	Shoulder height (m)	Wave height (m)	Wave period (s)	Wave force (kN/m)	Force height (m)	Dynamic uplift (kN/m)
II	No	5	9	1036.3	9.48	525.1
III	+6	5.91	12.7	1436.1	10.36	878.2
IV	+11	8.04	12.7	748.9	6.10	766.2

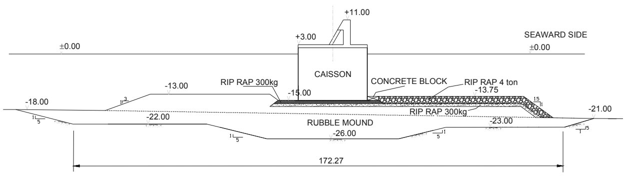

Figure 5. Design of the caisson breakwater.

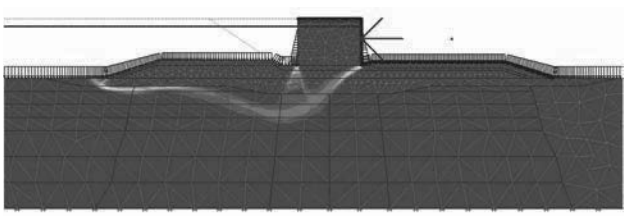

Figure 6. Failure mechanism for Phase III under storm loading.

4 CYCLIC SHEAR STRENGTH

However, breakwaters are also subjected to cyclic wave loading. Storms are the primary source of energy that may cause cyclic mobility or, in extreme conditions, liquefaction of foundation soils. Even if such extreme events do not occur, undrained shear strength may be lower after a severe episode of cyclic loading. Consequently, clay behaviour under cyclic loading was also investigated in the laboratory by performing cyclic simple shear and triaxial tests (on isotropically and anisotropically consolidated specimens). Data from simple shear tests were favoured because they appear to correspond more closely to the actual breakwater foundation conditions during storm loading.

Results from these tests can be usefully summarized using interaction diagrams such as that shown in Figure 7. This diagram shows a relationship between the normalized average shear stress τ_a/σ'_{vc}, normalized cyclic shear stress τ_{cy}/σ'_{vc} and the number of cycles to reach the cyclic mobility criteria. Also, results obtained from simple shear testing on the plastic Drammen clay (Goulois et al, 1985) are shown for reference.

Failure occurs for a given combination of normalized cyclic and average shear stress. Figure 7 shows the approximate bounds of these combinations for two different loading conditions (40 impacts and 1000 impacts). The normalised cyclic shear stress τ_{cy}/σ'_{vc} , for low values of the normalized average shear stress, is close to 0.17 for 40 cycles and to 0.10 for 1000 cycles. A second static bound is provided by the relationship $\tau_{cy}/\sigma'_{vc}+\tau_a/\sigma'_{vc}=0.25$, which is based on the previous discussion on static undrained strength.

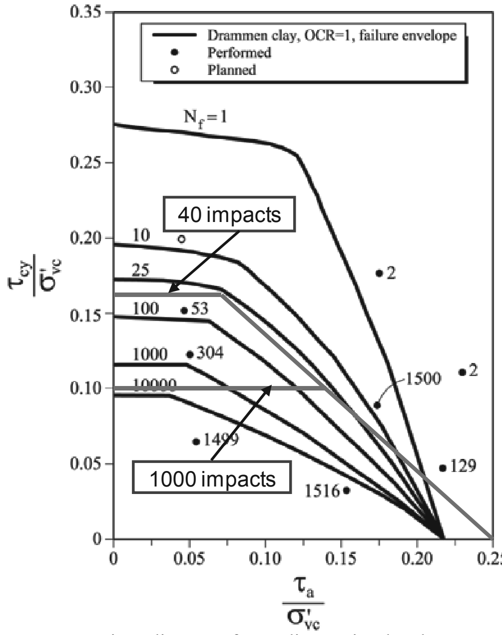

Figure 7. Interaction diagram from direct simple shear tests (NGI, 2002).

5 SIMPLIFIED ANALYSIS USING THE INTERACTION DIAGRAM

An example of the simplified stability analysis concerning Phase III of construction is presented in this section. The design storm established for this Phase is summarised in Table 2 that contains the number of waves of different heights corresponding to a succession of storm intensities with different durations and different significant wave heights.

It is assumed that the design storm can be represented as the application of a number of wave impacts of a certain magnitude. Then, a static analysis can be used to identify areas in the foundation soil where the stress state exceed the criteria of unstable stress defined by the interaction diagram. Naturally, to use the information contained in the interaction diagram, it is necessary to transform the variable wave loads of the storm into a series of cycles of uniform magnitude. This transformation always involves, to a certain extent, a degree of uncertainty and approximation. It is therefore advisable to adopt a measure of conservatism.

With this approach, two loads intensities were selected from the wave magnitudes shown in Table 2: a large load of 1011.5 kN/m and a smaller load of 341.6 kN/m. The former is assumed to act forty times and the latter five thousand times. The limit criteria corresponding to those two numbers of cycles have been indicated in Figure 7.

Now, it is possible to compute, using a conventional static finite element analysis and applying the corresponding wave loads, the points at which such criteria are exceeded, indicating the possibility that, in those zones, a degree of cyclic mobility occurs with a potential reduction of the undrained shear strength. A quite conservative assumption is that the operational undrained shear strength reduces to the residual value of c_u. The foundation zones affected are shown in Figures 8 and 9. They are quite similar in the two cases and affect only a quite limited area of the foundation soil.

Table 2. Characteristics of the design storm

Wave height (m)	Number of waves H_s = 3m (24 h.)	H_s = 4m (24 h.)	H_s = 5m (24 h.)	H_s = 5.9m (24 h.)	Wave force (kN/m)	Force height (m)
1-2	3124	1661	593	150	213.4	9.77
2-3	2203	1693	720	199	341.6	10.28
3-4	848	1133	626	198	475.0	10.62
4-5	195	545	427	161	685.0	10.65
5-6	28	194	236	111	825.4	10.74
6-7	2	52	108	66	870-5	10.80
7-8	0	11	41	34	920.0	10.00
8-9	0	2	13	16	1011.5	10.36
9-10	0	0	3	6	1410.1	11.46
10-11	0	0	1	2	1528.0	11.15
11-12	0	0	0	1	1559.3	11.38

Finally, a new stability analysis is performed with the new distribution of undrained shear strength of the foundation soil for the two cases considered. The analysis also considers the influence of the dynamic uplift caused by the storm loading, derived from the physical model tests carried out for this particular breakwater design. The results, in terms of factor of safety for strength reduction, are shown in Table 3. It can be seen that consideration of cyclic loading has a moderate but noticeable impact on the factor of safety. Given the exceptional character of the design storm and the conservative assumptions made in the analysis, the factor of safety obtained was considered adequate for accepting the design.

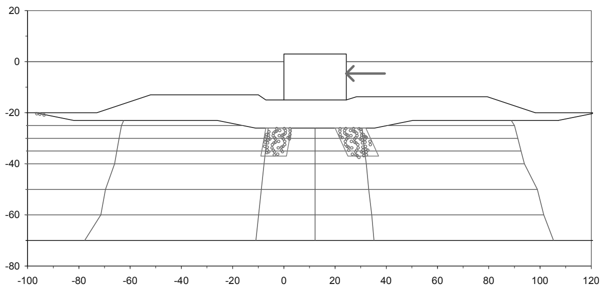

Figure 8. Foundation zones exceeding the interaction diagram criterion. Wave load = 341.6 kN/m and 1000 cycles.

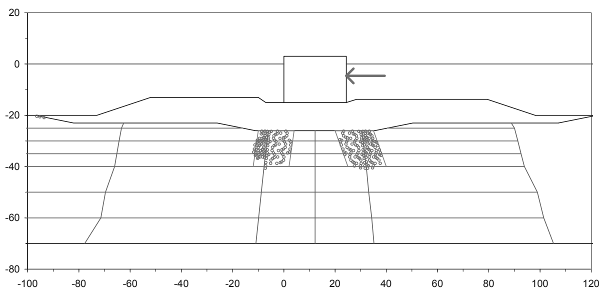

Figure 9. Foundation zones exceeding the interaction diagram criterion. Wave load = 1011.5 kN/m and 40 cycles.

Table 3. Computed factors of safety (on strength parameters)

Consideration of cyclic loading	Factor of safety: Wave force = 341.6 kN/m 1000 impacts	Wave force = 1011.6 kN/m 40 impacts
Yes	1.48	1.18
No	1.55	1.40

6 CONCLUDING REMARKS

A key design feature of a breakwater is the assessment of the stability of the breakwater when subject to extreme storms. This is particularly the case for caisson breakwaters in which the effects of wave action are significantly stronger than for the classical rubble mound type. A proper consideration of the dynamic effects would require the performance of a full dynamic analysis. Here, a simplified stability analysis is proposed that takes into account the potential reduction of the shear strength of the soil due to cyclic loading. It is based on the experimental determination of the interaction diagrams that provide criteria to identify the conditions for which the soil can undergo cyclic mobility and strength degradation. The corresponding strength reduction is then taken into account in conventional stability analyses. The procedure has been illustrated by a specific application to a caisson breakwater that is part of the extension works of the Barcelona Harbour.

7 ACKNOWLEDGEMENTS

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