CHAPTER 101

REDUCTION OF WAVE FORCES AND OVERTOPPING BY SUBMERGED STRUCTURES IN FRONT OF A VERTICAL BREAKWATER.

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1.- ABSTRACT:

This paper describes some of the results obtained from a series of model tests to measure wave forces and overtopping in a vertical breakwater located in a surf zone. The aim of the tests was to better design conditions using submerged structures. With the sections proposed in this study, it is possible to achieve a considerable reduction in wave forces and overtopping on vertical breakwaters.

2.- INTRODUCTION

Most vertical breakwaters in Spain have been constructed at depths where breaking does not occur. It is a well known fact that the wave pressure to which a vertical breakwater is subject in breaking conditions, is much greater than when such conditions are absent, so the design conditions vary considerably. As a result of this, some solutions, including vertical breakwaters with block mounds, have been developed with a view to dissipating energy. Nevertheless, these solutions have not taken into account their possible use for recreational purposes, such as Marinas; in these cases, the question of aesthetics is fundamental as regards the impact that they might have on the coastal environment.

Recently, the C.E.P.Y.C. has carried out tests on a series of sections subject to irregular waves, with a view to improving design conditions for a vertical breakwater situated in a surf zone. The various options studied, are

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based on the geometrical variation that crowns the caisson and on the inclusion of submerged elements in front of the vertical face breakwater, which modify the conditions of the incident wave. These elements consist of a submerged breakwater and rubble toe protection with a small rubble fill between them.

These tests were made for the Marina breakwater at the Olympic Village in Barcelona, whose technical engineers proposed different test solutions. In addition to this work, a series of complementary tests were carried out in order to obtain further information concerning the overtopping rate.

3.- TEST CONDITIONS

The tests took place in a tank with an overall length of 45 m, 6·5 m wide and 0·80 m deep, at the C.E.P.Y.C. laboratories in Madrid.

A channel 1·3 m wide was placed inside the tank, into which the test section was located, facing the normal direction of the wave propagation generated by the paddle.

The depth of the water at the foot of the paddle was 0·70 m, so as to guarantee wave generation in acceptable conditions, and the depth at the foot of the test section was 0·225 m, with a 1:30 gradient at the bottom. A 1:40 scale was used in the tests.

A dynamometer was used for measuring the wave forces, whose ranges of measurement were:

Fx and Fy	: 40 Kp	(Model units)
M	: 600 Kp.cm	(Model units)

As regards the measurement of the overtopping, which included: the amount of these expressed in terms of the number of waves contained in the wave recording, the height reached by the "sheet" of water above the crown and the volume of water that overtopped, we would say that, the recordings of the first two cases were carried out using a device designed in the C.E.P.Y.C., consisting of a transductor made up of a fibreglass plate on which a printed circuit was attached, this being connected to a series of electrodes 1 cm apart. As the water flowed over them, the circuit closed and thus provided the overtopping height (in centimetres) as a sum of the wet electrodes, the abovementioned overtopping also being counted in the overall calculation. The measurement of the volume of water which overtopped was achieved, by a tray attached to the rear part of the section. The results of this volume are given per linear metre of section of breakwater and per second (m**3/m/s).

The output of the analogical signals from the dynamometer and the overtopping gauge are conditioned afterwards, by means of an A/D convertor and digitalized so that they can be read by a computer. A description of of this process is shown in Figure 1.

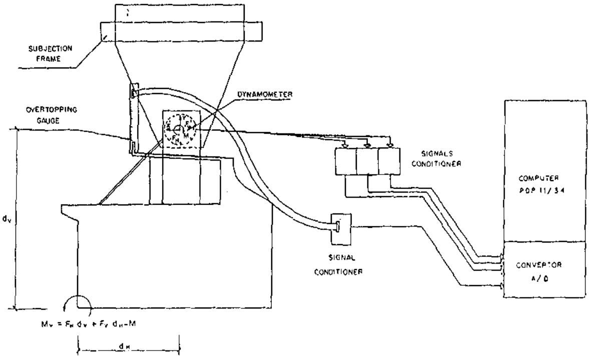


Figure 1. Measures diagram

The tests were carried out in three types of storm conditions, adjusted to a theoretical spectrum of the J.O.N.S.W.A.P. kind, whose characteristics are similar to those that occur in the Spanish Mediterranean; the wave conditions of these storms, are shown in table 1.
Where:
Gamma is the peak enhancement factor of the Jonswap spectrum, Tp is the peak period of the wave spectrum in seconds and Hs is the significant wave height in metres. The callibrated test waves correspond to records of 300 to 325 waves.

STORM	Tp(s)	Hs(m)	GAMMA
I	10	6, 7.5	3·3
II	12	6, 7.5	3·3
III	14	6, 7.5	3·3

Table 1.

On considering the depth of the toe of the structure d = 10·20 m, the bed slope m = 0·033 and the three peak periods of the storms, it could be concluded that the structure under the different wave calculations, would be subjected to unbroken waves, broken waves and breaking waves, critical conditions being given for waves that break or which are at breaking point over the vertical facing.

4.- PROCEDURE

The sequence of tests was developed in four distinct stages, each one constituting an approximation to the optimization of the section for the design wave conditions

The First Stage, consisted of testing three vertical breakwaters with the same crown height hc = 8·4 m (which did not vary during the different test stages), but having different crown shapes, these included solutions 1,2,3. Figure 2.

The storms that these structures were subjected to were storms I and III, shown in Table 1.

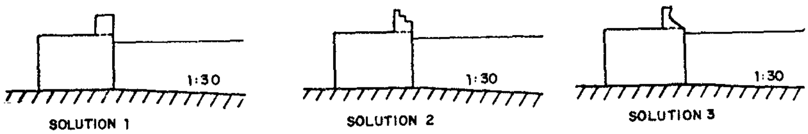


Figure 2. First stage solutions

The second stage consisted of placing a submerged breakwater that would serve as a filter to the incident wave, causing it to break, so that the wave forces and overtopping at the vertical breakwater would be reduced, Figure 3. initially, the solutions tested were 4, 5 and 6, that vary as regards the distance between the transversal axis of the submerged breakwater and the wall of the vertical breakwater (L) and the crown height of the submerged breakwater (Cc). After this, and with a view to observing the degree of operativity of the combination vertical breakwater - submerged breakwater, complementary overtopping measurements were carried out with the solutions 4a, 4b, 4c, 5a and 6a, whose characteristics can be seen in Table 2.

The wave conditions tested, corresponded to storms II and III in Table 1, with steps for significant waves of 4, 5, 6 and 7·5 metres.

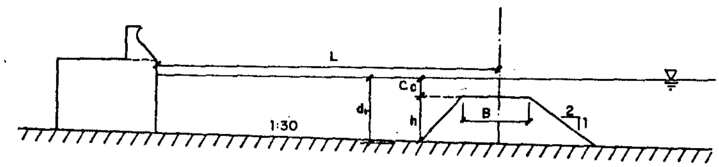


Figure 3. Second stage solutions

SOLUTION	B(m)	Cc(m)	L(m)	dr(m)	ct	tp	Fill
1	-	-	-	-10.2	1	-	-
2	-	-	-	-10.2	2	-	-
3	-	-	-	-10.2	3	-	-
4	18	-3.5	60	-10.2	3	-	-
4A	18	-2.5	60	-10.2	3	-	-
4B	12	-3.5	60	-10.2	3	-	-
4C	12	-2.5	60	-10.2	3	-	-
5	18	-3.5	85	-10.2	3	-	-
5A	12	-3.5	85	-10.2	3	-	-
6	18	-2.5	85	-10.2	3	-	-
6A	12	-2.5	85	-10.2	3	-	-
7	18	-2.5	60	-10.2	3	Y	-
8	18	-2.5	60	- 6.0	3	Y	Y
9	18	-2.5	60	- 4.0	3	Y	Y
10	18	-1.5	60	- 4.0	3	Y	Y
11	18	-1.5	60	- 6.0	3	Y	Y
12	30	-2.5	66	- 6.0	3	Y	Y
13	30	-1.5	66	- 6.0	3	Y	Y
14	30	-1.5	66	- 4.0	3	Y	Y
15	30	-2.5	66	- 4.0	3	Y	Y
16	18	-2.5	60	- 6.0	2	-	Y
17	30	-2.5	66	- 6.0	2	-	Y

TABLE 2.

The Third stage was made up by solutions 7 to 15, and consisted of placing rubble toe protection at the base of the vertical breakwater. As regards the submerged breakwater, the geometrical parameters were varied: Cc, L and B, as well as considering rubble of 8 Tons and different parallelepiped concrete block weights on the crown. Fillings of different depths (dR) were tested between the submerged breakwater and the attached rubble, Figure 4 and Table 2.

The test storms were those referred to as II and III in Table 1.

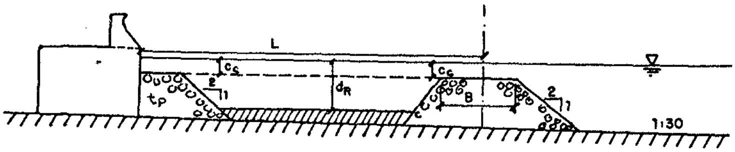


Figure 4. Third stage solutions

The Fourth Stage, comprised solutions 16 and 17. It consisted of testing a tiered superstructure of the type used in solution 2, but occupying the whole width of the caisson. Likewise, the protective rubble attached to the vertical breakwater was removed, one sole thickness of filling being considered, the variation between the two solutions referred to being determined by the crown width of the submerged breakwater, B, Figure 6 and Table 2.

The test waves at this stage, were storms II and III from Table 1 with step heights for significant waves of 4, 5, 6 and 7·5 metres.

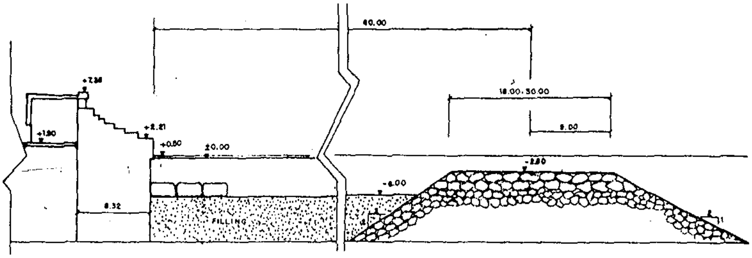


Figure 5. Fourth stage solutions

5.- WAVE FORCE ANALYSIS

The analysis of the results as far as the wave forces is concerned, will only be carried out for the maximum horizontal force recorded by the dynamometer throughout the length of each one of the tests, these being characterized by significant wave height and a specific peak period.

The analysis will be made for the solutions included in each one of the stages outlined in section 4. and, finally all the solutions will be compared as a whole.
The maximum horizontal forces of the first group of tests, corresponding to the first three solutions, can be seen in Figure 6.

The results clearly show, a reduction in the resulting force, undergone in solutions 2 and 3, that at the height +2 present a sloping and tiered surface at the crown of the section, by contrast to solution 1, which has a totally vertical surface. At the same time, the influence of the peak period in the value of the wave force, is clearly shown.

The reduction referred to, as is known, is a result of the lag that exists between the wave action on the two surfaces of the structure; i.e.: that of the caisson, totally vertical, and the tiered sloping surface of the crown. Likewise, the discontinuity that is implied in the sloping plane, also prevents high pressures, known as impulsive breaking wave pressures, from being brought to bear, because they occur slightly above the still water level (S.W.L.); however, this kind of pressure does feature in solution 1, as all the necessary conditions are present.

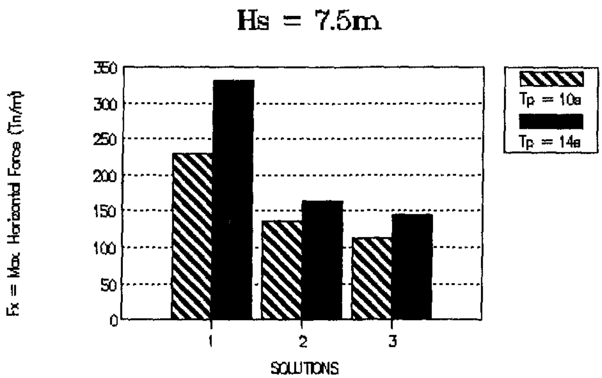


Figure 6. Results fron first stage.

Mitsuyatsu (2) demonstrated, on the basis of a series of systematic experiments that, where there is no rubble mound breakwater foundation and as long as the breaking of the wave occurs in front of a vertical wall, impulsive breaking wave pressure could be brought to bear if the sea bed were equal to or greater than 3% and the wave incidence angle were below 20º, as is the case in solution 1.

Nevertheless, in spite of the considerable reduction in the wave force in solutions 2 and 3, as compared to sollution 1, neither of the two can be considered acceptable from the viewpoint of values of the safety factors required for the sliding and overturning of a particular caisson, this having been previously designed. So various options were designed in front of the breakwater and this gave rise to the subsequent stages that were commented upon in section 4.

Solutions 4, 5 and 6 made up the second stage of the present work. The results, as far as the maximum horizontal force is concerned, are shown in Figure 7.

The differences that exist between the results, are determined by the variation in depth of the crown of the submerged breakwater (Cc) and by the length (L) between the vertical face of the caisson and the axis of the submerged breakwater.

It can be observed from the comparison of results, how length (L) is the most influential of the abovementioned parameters, while Cc, at least as regards the experimented values (a difference of 1 m), hardly undergoes any variation, if the respective values of solutions 5 and 6 are observed.

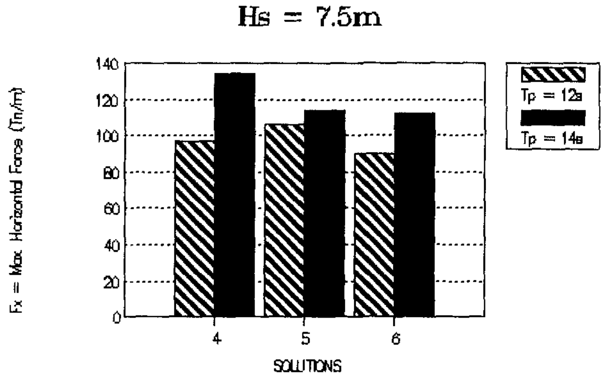


Figure 7. Results from second stage

The reduction in the maximum horizontal force, as a result of the submerged breakwater, in the three solutions, with respect to solution three in the first stage, this serving as a reference, is 8·3 %, solution 4, 18 %, solution 5 and solution 6, 22·75 %. In spite of the reduction in two of the cases being significant when compared to solution 3, it still does not meet the safety factor values that are required for the specific caisson.

The analysis of the next group of tests, belongs to the third stage, and the different solutions are characterized, as was mentioned in section 4, by the variation in some of the geometrical parameters of the submerged breakwater, as a result of a rubble toe protection at the vertical breakwater and by a filling between both breakwaters: vertical and submerged.

The results of this stage can be seen in Figure 8. It can be said that the filling at height -6 m, barely influenced the dissipation of the incident wave energy, once this passed over the submerged breakwater, which can be seen from the values of the figures that correspond to solutions 7 and 8; nevertheless, the same filling, increasing the crown height by 2 m to -4 m, brought about a reduction that lies in a range that goes from 8·5 % to 14 % (compare with solutions 8-9, 13-14 and 12-15).

Solutions 10, 11, 13 and 14 were tested by placing one single layer of parallelepiped elements of concrete blocks of different weights on the crown of the submerged breakwater; as regards this, it could be said that solutions 10 and 11, with respective weights of 20 Tn and 25 Tn, gave worse results than those others where 8 Tn quarry stone units were used on the crown, through the absence of interlocking between the concrete units and, as a result, there was an increase in damage once the first test waves attacked: this caused a reduction in the crown height.

45 Tn elements were placed for solutions 13 and 14. Such a great weight difference gave rise to a decrease in the damage and caused an improvement in behaviour, as far as the recording of maximum horizontal force was concerned, and when compared to solutions 10 and 11, although the crown height width went up from B = 18 m to B = 30 m.

The influence of parameter B in the stretches being tested, can also be seen when comparing the results of solution 8 and solution 12, the latter undergoing a reduction in wave force of 16·6 %.

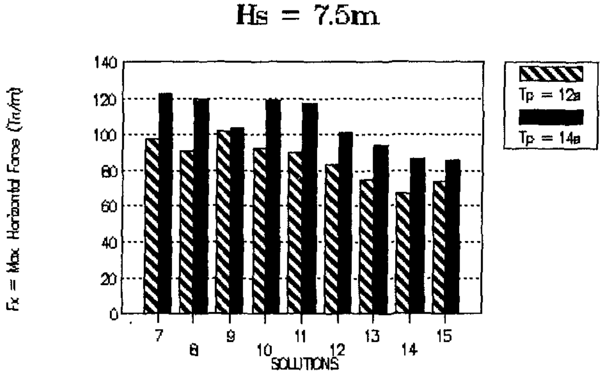


Figure 8. Results from third stage

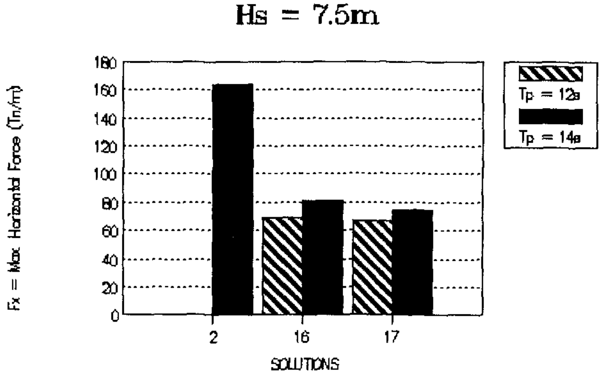
In the last solutions to be tested, 16 and 17, the configuration of the caisson superstructure was changed, its profile being completely tiered, as can be seen in Figure 5. In addition to this, the rubble attached to the caisson toe was removed, the filling being retained.

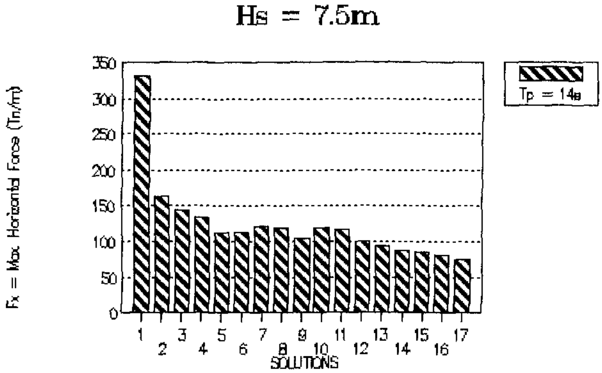
The results of these solutions can be seen in Figure 9. The result of solution 2, that corresponds only to the caisson with tiered superstructure, is also shown. The reduction of the maximum horizontal force exceeds that of solution 2 in both, and by more than 50 %. The different crown widths of the submerged breakwater, B = 18 m and B = 30 m that were tested, determine the variation in the results of solutions 16 and 17.

In the light of these results, solution 16 was chosen as the definitive one, that on the basis of the maximum simultaneous wave forces recorded on the dynamometer, complied with the advisable and authorised safety factors in the practice, in accordance with the caisson width that had been previously adopted.

Finally, Figure 10 shows all the results of each one of the solutions tested, corresponding to the maximum horizontal force reached in every one of them.

By mere observation, the reduction that occurs when comparing solution 1 with solution 16, can be considered spectacular, to say the least.





Figures 9 and 10

6.- WAVE OVERTOPPING ANALYSIS

The measures that were taken as regards wave overtopping were; firstly the number of waves that overtopped the breakwater (N), with respect to the total number of waves in the recording (Overtopping %). Secondly, the height of the sheet of water that overtopped the crown of the breakwater, to which, if we add the distance from the water level of the design to the crown height, can be called "overflow wave run-up", from whose values the parameters R1/3, R and Rmax are obtained, through the statistical analysis of the recordings. Finally, the total volume of overtopping during the test storm, expressed in terms of one m3/s per running metre of vertical breakwater, (average overtopping rate (Q)).

The most important factor for the design engineer concerns the value of Q, because this parameter is the one that determines the flood level behind the breakwater, the capacity that the structure's drainage system requires or the degree of operativity during the storms, of the installations, behind the breakwater.
Making an overall analysis of the overtopping rate obtained for the different solutions tested, we could say that the inclusion, in front of the breakwater, of the different submerged elements, with their respective test sizes, has proved to be effective, as regards the reduction of the overtopping rate; this can be observed in Figure 11, which refers to the most unfavourable wave conditions.

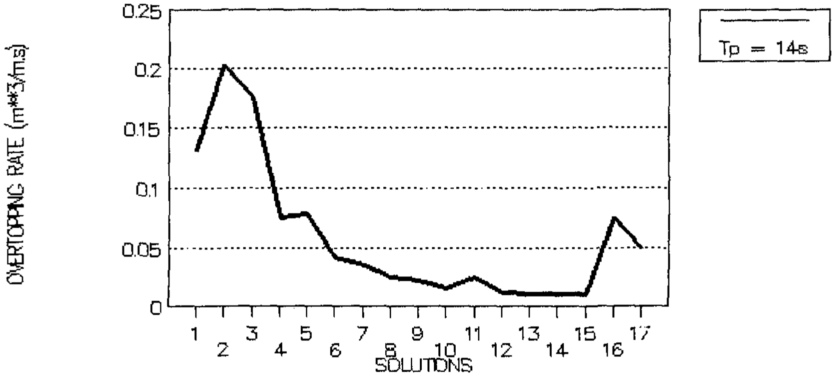


Figure 11. Overtopping comparative graphic, Hs=7.5m

It is not possible to give design recommendations in this work, because in order to do this, it would be necessary to take the geometrical variables of the different element into account, together with a greater number of tests, but, on making an analysis by stages, it is possible to talk of the different effects that could be observed under test conditions.

The first stage of the tests had as main aim to observe the effects, in the wavw run-up and overtopping, of the three shapes of crown that were chosen. In figure 12, it can be seen that the least favourable crown was the one used in solution 2. As regards the wave run-up, it can be clearly stated that there are no great differences between the solutions, it turning out to be a very uniform phenomenon for the two test storms (Tp = 10s and Tp = 14s), nevertheless, the final difference in the results is marked by the overtopping percentage that is found in line with the overtopping rate.

Of the different solutions tested at the second stage complementary tests, i.e. solutions 4, 4a, 4b, 4c, 5, 5a, 6 and 6a, see Table 2, firstly it can be said that, compared to the previous stage, reductions have been obtained in the wave run-up and overtopping through the use of the submerged breakwater, Figure 11, and secondly, that for the two phenomena, the least favourable storms were those characterized as Tp = 14s, Figures 13 and 14.

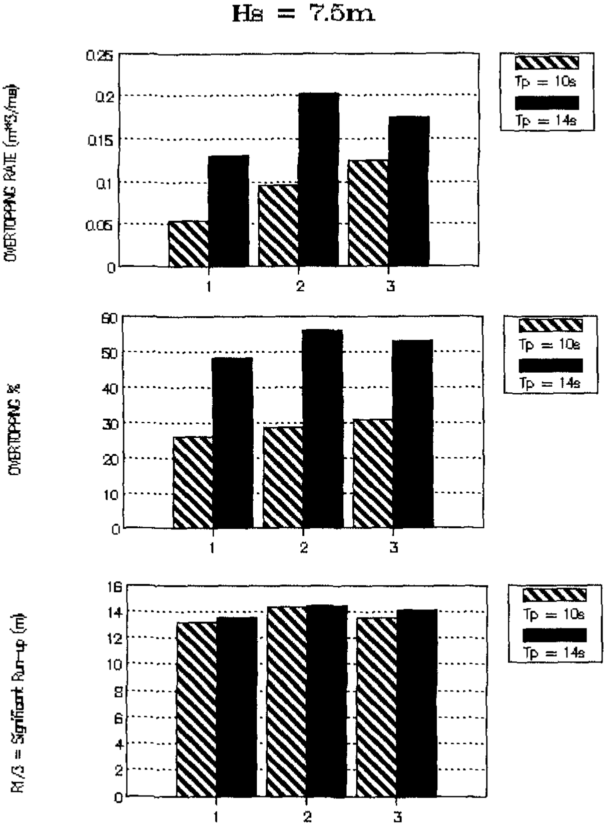


Figure 12. Results from solutions 1,2,3 with Hs=7.5m

Thirdly, that, within the magnitude of the values tested in variables L, Cc and B, the run-up phenomenon still shows very uniform behaviour, the best results being observed at this stage for L = 60 m and B = 12 m. However, in the case of the overtopping rate, it can be seen that parameters Cc and L do not influence very much, and that it is parameter B, for the solutions that have a greater value (B = 18 m), which gives the best results both for Tp = 12s and Tp = 14s. Figures 13 and 14.

At the third stage, owing to the fact that the run-up phenomenon is still fairly uniform and unhelpful in the evaluation of the various solutions, it will not be taken into account in this analysis.

On studying the results of the different solutions at this stage (7 to 15) with respect to the overtopping rate, we can say that, simply by including the protection at the toe of the vertical breakwater acting together with the submerged breakwater, solution 7, Figure 15, reductions are obtained in the overtopping with respect to the two previous stages, and then, on reducing the existing depth in the space formed between the two breakwaters using a rubble filling, we continue to obtain improvements the lesser the depth (dR = -10.2,-6,-4). Nevertheless, the most influential variable is still the width of the berm (B) of the submerged breakwater, a considerable improvement being observed when it is increased to 30 m.

Finally, of the solutions taken at the 4th stage, it can be seen from Figure 11, how the omitting of the wall protection influences the results, and, also the influence of the width of the berm (B) between 18 m and 30 m.

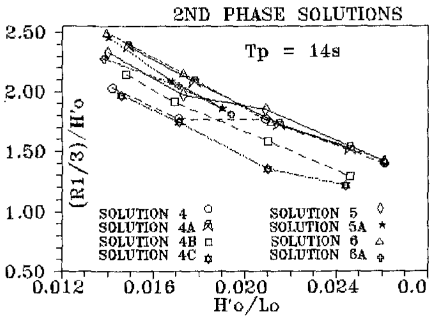


Figure 13. Wave Run-up

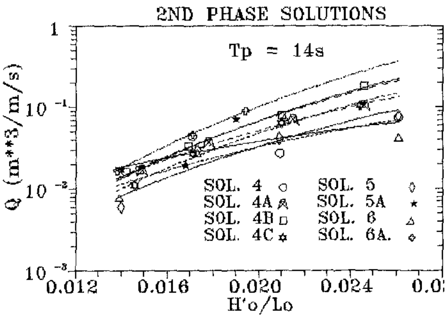


Figure 14. Overtopping

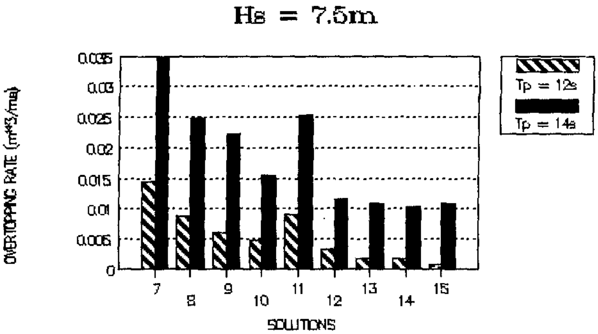


Figure 15. Results from third stage solutions

7.-CONCLUSIONS

The different activity, carried out in front of the vertical designed breakwater, have been shown, at the Marina of the Olympic Village for Barcelona-92, they were designed to optimize the abovementioned breakwater from the point of view of its stability and functionality.

In the light of the tests performed, it is not possible to establish any design criteria to determine the optimum geometrical characteristics of the submerged breakwater situated in front of the vertical one; it would only be possible by the realizing of a larger number of more systematic tests carried out in order to detect the sensitivity of each of the parameters involved in the wave attack (Cc, B, L and dr).

Nevertheless, it can be said that, within the range of values experimented for the above-cited parameters, the crown width of the submerged breakwater, B, turned out to be the most sensitive to both the wave force and the overtopping.

As far as the "overtopping/run-up" is concerned, it can be said that the statistical parameter R1/3, used in its analysis, hardly showed significant variations for the different situations tested.

8.-ACKNOWLEDGEMENTS

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