

CHAPTER 138

THE DESIGN OF A SLOTTED VERTICAL SCREEN BREAKWATER

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INTRODUCTION

This paper describes the wave studies, model testing and structural design of a 250 metre long slotted vertical screen breakwater for a marina at Plymouth, England.

The marina is being developed by Plymouth City Council to provide a purpose built facility for hosting the major long distance races that start or finish at Plymouth, in addition to the usual marina facilities for private boat owners.

After examining three alternative locations Plymouth Council selected a site in the north-east corner of Plymouth Sound as shown in Figure 1.

The site is confined on two sides by existing shipping channels and by a rocky shore on the land side. Exposure to wave attack is limited to the south west sector.

Because the existing seabed at the marine site was the responsibility of the Duchy of Cornwall, an Act of Parliament was required before Plymouth Council could commence construction. The Act contains clauses regulating the use of the marina and the permissible changes to the wave conditions in the adjacent shipping channels.

DESIGN CRITERIA

The basic maritime criteria governing the design are:-

- (i) Wave heights in the marina should not exceed $H_s = 0.3$ metres for more than 200 hours per year.
- (ii) With the exception of a 10 metre carriageway on the SW face of the breakwater wave heights in the adjacent shipping channels should not be increased, as a consequence of the breakwater, by more than 50% above the conditions existing before construction.
- (iii) The breakwater should be designed to withstand the 100 year return period storm occurring at any tide level.

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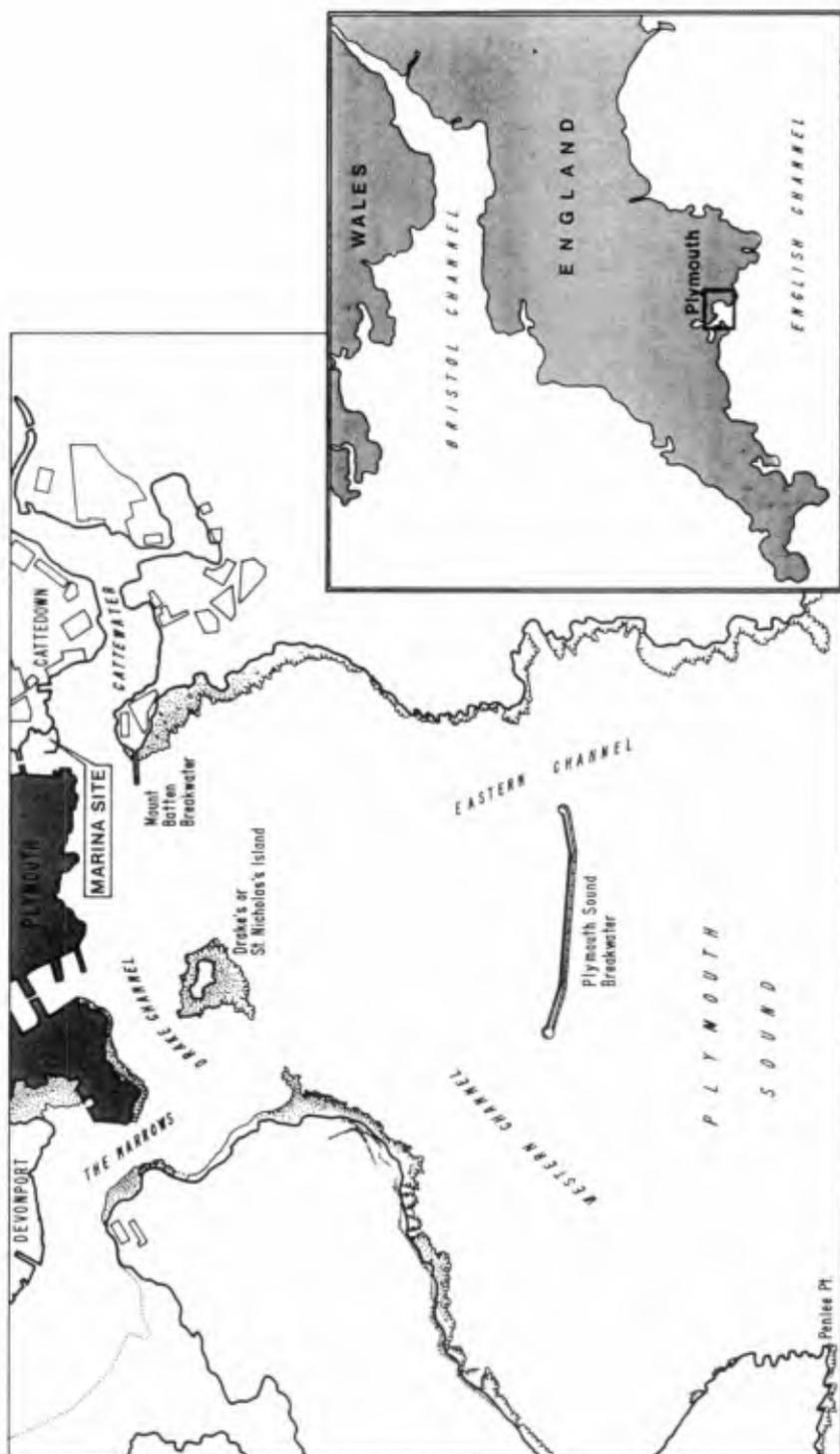


Figure 1: Site Location Plan

The preparation of construction drawings and tender documents was not started until parliamentary approval had been given. This was obtained in April 1985 and Plymouth Council decided that the marina should be ready for hosting the Two Handed Trans Atlantic Race in June 1986. To achieve this programme detailed design, testing and tender documents had to be prepared in the period May to mid-August so that tenders could be received by mid-September to allow construction to commence in November 1985.

EXISTING WAVE CONDITIONS

Deep water swell from the English Channel can only reach the marina area from the south-west. The most severe wave conditions will therefore comprise combined swell waves and locally generated wind waves arriving from this direction.

Data for deep water wave conditions were available from the Institute of Oceanographic Sciences (IOS) in the form of a report on data collected by a Waverider buoy located near the Eddystone Lighthouse. The deepwater wave climate was transferred to the marina site in two stages.

- (i) wave refraction backtracking and spectrum transfer from deep water to the Plymouth Sound breakwater.
- (ii) diffraction of the resulting wave energy spectrum from the Plymouth Sound breakwater to the marina site.

Wave heights and periods for locally generated waves were calculated using the Carter Equations derived from the Jonswap energy spectrum formulation. Wind speeds calculated from data recorded at Fort Stanford, Plymouth and the equations used for a fetch limited growing sea were:

$$H_s = 0.0163 X^{0.5} U$$

$$T_z = 0.439 X^{0.3} U^{0.4}$$

where X is the effective fetch in kilometres and U is the wind speed in metres per second.

The resulting wave heights at the marina site were as follows:

Return Period	Hs(m) (deepwater)	Hs(m) (local)	Hs(m) (combined)
100	1.5	0.8	1.7
50	1.4	0.75	1.6
10	1.2	0.7	1.4
1	1.1	0.6	1.3

Examination of the Admiralty Charts for Plymouth shows that the approach direction for the swell waves is limited by Penless Point to the west and the Plymouth Sound and Mount Batten Breakwaters to the east. Approach directions for the critical wave condition are therefore limited to the sector between 210° and 220°.

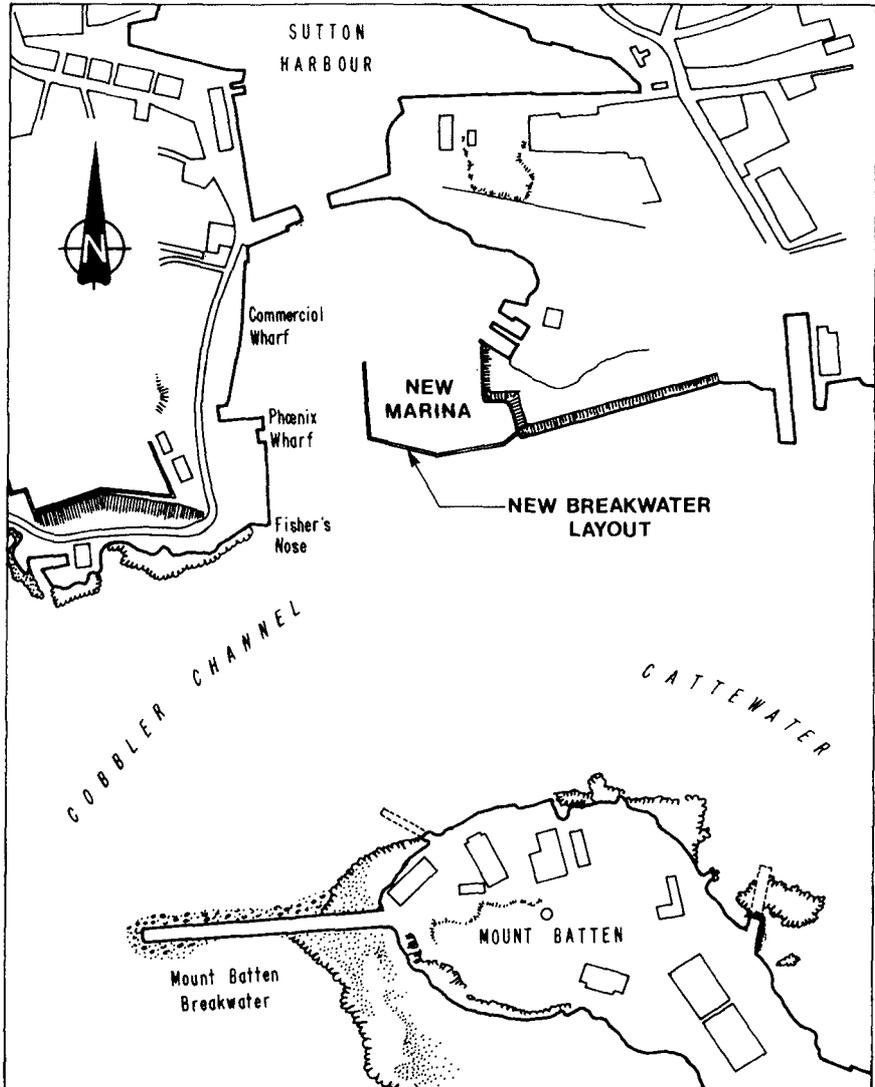


Figure 2: Outline plan for new marina

Under critical wave conditions the wave climate to the west and south of the marina will be influenced by sheltering and diffraction effects caused by the coastline adjacent to Fisher's Nose (see Figure 2).

The treatment of wave diffraction was based on the Sommerfeld solution as presented by Wiegel (1964). A Halcrow computer model was used to evaluate wave height factors at the nodes of a square grid across the area of study. Wave height factors being defined as:

$$\text{wave height factor} = \frac{\text{wave height at node}}{\text{height of incident wave}}$$

The model could be used to evaluate wave conditions in the vicinity of systems of one or two impermeable breakwaters. The breakwaters could also assume any degree of reflection of incident wave energy.

The sheltering effect of Fisher's nose was represented by a breakwater aligned north/south. The resulting wave height factors are given in Figure 3 where the factors are expressed relative to the incident wave height off Fisher's Nose.

WAVE CONDITIONS AFTER CONSTRUCTION OF THE MARINA BREAKWATER

The wave diffraction model was run for each segment of the breakwater and the results were superimposed to provide a composite diffraction solution. Thus a relatively simple program provided a quick and realistic answer. The results for a fully reflecting breakwater are shown in Figure 4.

Comparison of Figure 3 with Figure 4 gives the percentage change in wave height factors resulting from construction of the breakwater. These initial results showed that wave heights would increase to an unacceptable level in the two adjacent shipping channels.

The wave diffraction model was re-run using breakwater reflection coefficients less than unity and it was found that acceptable wave conditions could be achieved with the following reflection coefficients:

$$\begin{aligned}\text{Section A} &- K_R \leq 0.8 \\ \text{Section B} &- K_R \leq 0.5 \\ \text{Section C} &- K_R = 1\end{aligned}$$

In this appraisal the consequence of varying the incident wave period was also examined and this was found to alter the area influenced by the reflected waves quite significantly.

CHOICE OF STRUCTURE FOR THE MARINA BREAKWATER

Several types of structure were considered during the feasibility stage of the project. These included: a floating breakwater; a rubble mound breakwater; a blockwork wall and a vertical screen breakwater.

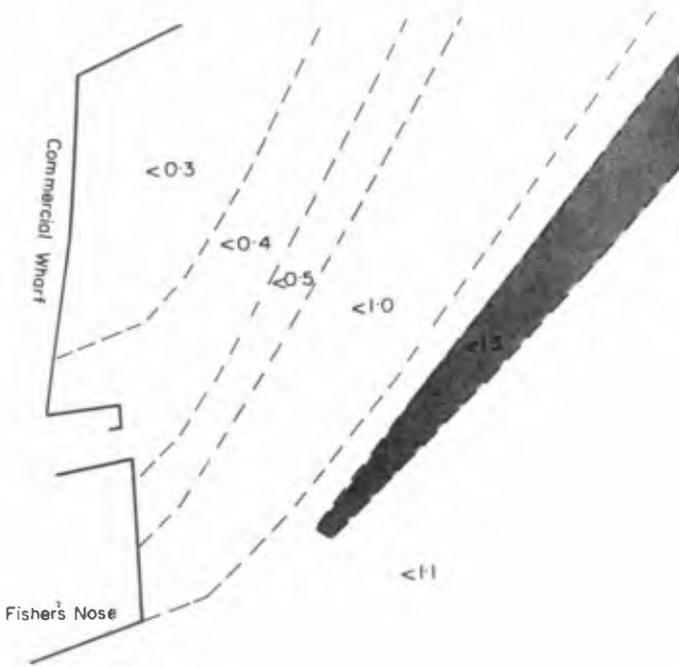


Figure 3: Wave height factors for existing site

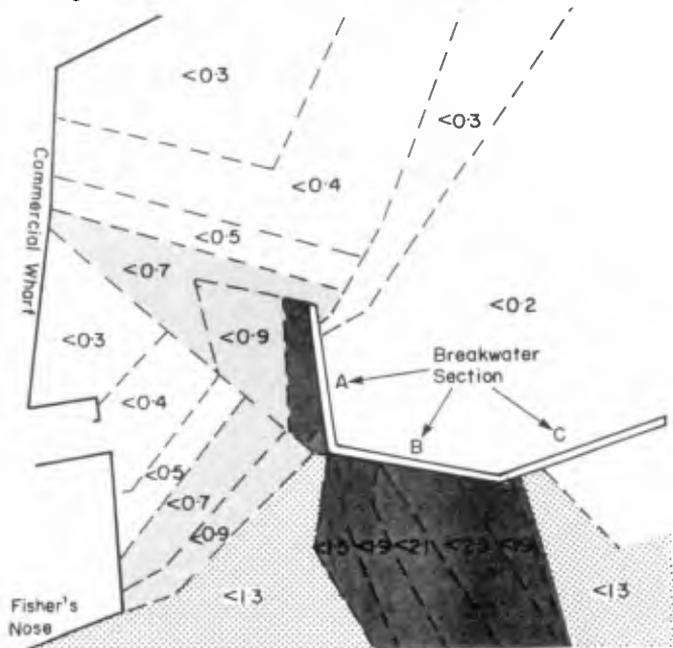


Figure 4: Wave height factors with fully reflecting breakwater

However this site has three major constraints: the seabed consists of soft silt up to 30 metres thick overlying bedrock; the area available for the marina is small; the spring tidal range is 5 metres. A vertical screen supported by piles driven to bedrock was considered to be the most practical and economic solution within these constraints.

Following the initial numerical diffraction and reflection analysis various configurations of perforated screens were examined. Examination of existing literature (refs 1, 3, 4 and 5) indicated that a double screen would be required for Section B whereas a single screen would be sufficient for Section A. The extent of the double screen was a balance between wave reflection, wave transmission into the marina and construction cost.

Although existing case histories were sufficient to demonstrate that a solution could be found to suit the basic design criteria there was insufficient information to permit detailed design to be carried out. The final configuration of the screens was therefore based on the results of a set of simple flume tests.

MODEL TESTING

To meet the tight design programme, model testing commenced at the start of the detailed design phase. This then enabled the final structural design to take account of the test results.

Model testing was carried out at Hydraulics Research, Wallingford in a 45 metre long wind wave flume. The purpose of the tests was to measure the reflection and transmission coefficients (K_R and K_T) for single and double screens whilst changing the following variables identified during the conceptual design stage:

- (i) Wave periods between 3 and 12 seconds
- (ii) Screen porosity varying between 8% and 35%
- (iii) Space between screens varying between 5 metres and 15 metres
- (iv) Slots horizontal or vertical
- (v) The gap between the bottom screen and the seabed
- (vi) The effect of changing the plank thickness from 300 mm to 150 mm
- (vii) Screens tilted or vertical

The model scale was 1:15 and random waves were used. Wave measurements were made using three wave probes and incident and reflected spectra calculated using an analysis program. The method determines the reflection and transmission coefficients for each frequency considered in the incident wave spectrum.

As the model tests progressed some changes were made to the testing sequence taking account of the previous results and the requirements of the structural design, which was proceeding simultaneously, thereby avoiding tests unlikely to yield a useful solution. In all 54 tests were carried out and the results are given by Hydraulics Research Laboratory (1986).

The principal results are summarised in Figures 5, 6 and 7.

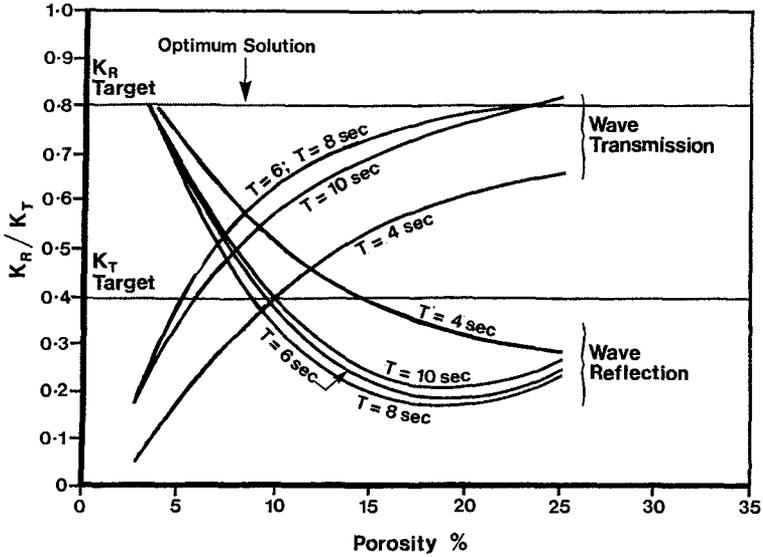


Figure 5: Reflection coefficient (K_R) & Transmission coefficient (K_T) v Porosity for single vertical screen

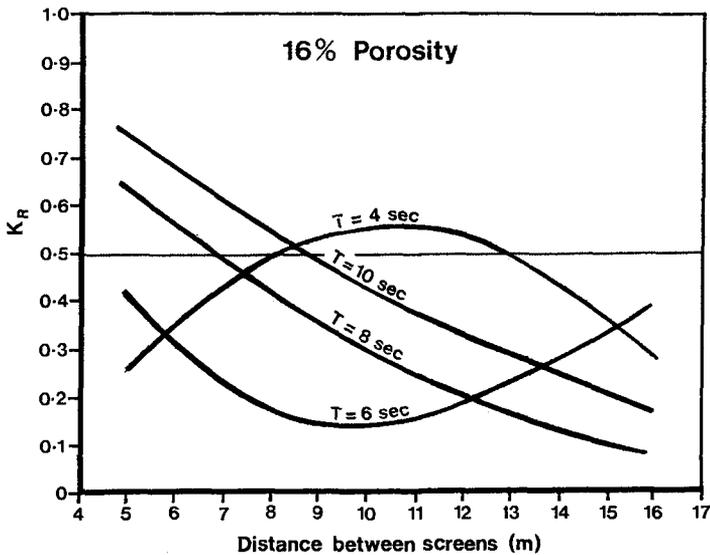


Figure 6: Reflection coefficient v Distance between screens

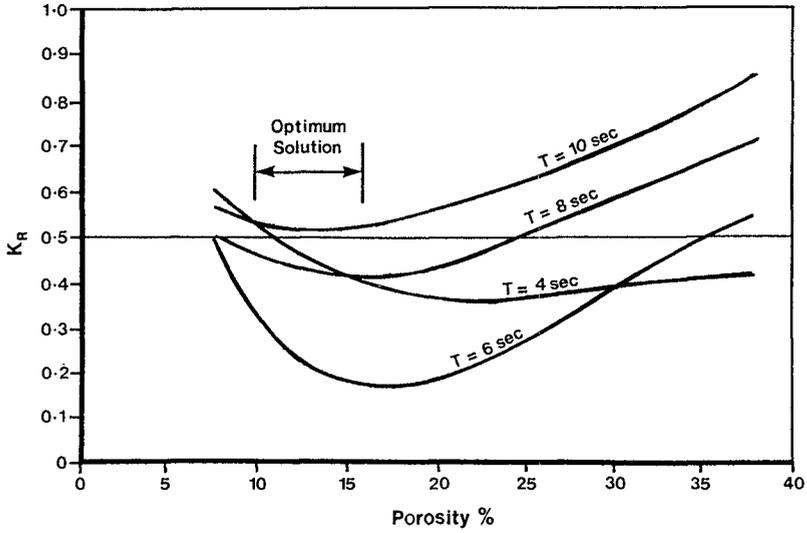


Figure 7: Reflection coefficient v Front screen details for screen spacing of 8m

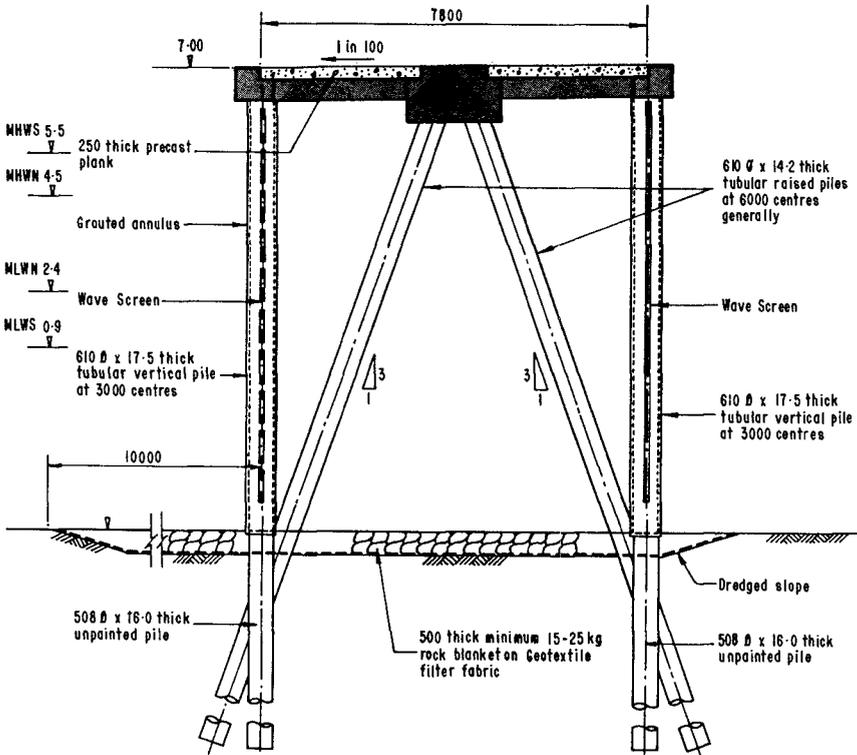


Figure 8: Section through double screen

For the single screen:

- (i) a porosity of 8% gave acceptable values of K_R and K_T (see Figure 5)
- (ii) the performance of horizontal slots was similar to that of vertical. Horizontal slots were therefore adopted because this orientation suited the proposed construction method
- (iii) a 1 metre gap beneath the screen gave unacceptable wave transmission results
- (iv) the hydraulic performance of 150 mm planks was similar to that of 300 mm planks.

For the double screen:

- (i) a double screen with a front screen with 16% porosity spaced 8 metres from a solid back screen gave suitable reflection coefficients (see Figures 6 and 7)
- (ii) the performance of horizontal slots was similar to that of vertical. Horizontal slots were therefore adopted because this orientation suited the proposed construction method
- (iii) a 1 metre gap beneath the screens gave unacceptable wave transmission results
- (iv) the hydraulic performance of 150 mm planks was similar to that of 300 mm planks
- (v) tilting the screens did not show any improvement over vertical screens.

FINAL WAVE STUDIES USING MODEL TEST RESULTS

The numerical diffraction model previously discussed was re-run using the results from the flume test as follows:

Wave Period (secs)	4		6		8	
Breakwater Section	K_R	K_T	K_R	K_T	K_R	K_T
A (single, 8% porosity)	0.6	0.3	0.6	0.4	0.6	0.4
B (double, 16% porosity 8 m space)	0.4	0.0	0.2	0.0	0.4	0.0
C (single, solid)	0.9	0.0	0.9	0.0	0.9	0.0

The results showed that wave heights outside the breakwater were within the required limits except for one small area to the south of the breakwater. It should be noted that the diffraction - reflection - transmission analysis does not take account of wave-wave interaction, wave breaking and the lateral internal diffraction of waves. As these non-linear effects would all lead to a reduction in resultant wave heights it was decided that this small area did not justify any changes in the proposed breakwater.

The results also showed that transmission into the majority of the marina had been retained below the target value of $H_s = 0.3$ metres. A small area in the south-west corner showing higher contours would be deleted by an additional length of double wave wall.

STRUCTURAL DESIGN

The typical structural details for the double screen are shown in Figure 8. The single screen is similar except that the rear screen and half of the deck is deleted. Typical isometric views of both types of screen are illustrated in Figure 9.

The wave loads on the screen are transmitted to the bedrock by the pairs of raking piles. The maximum compression and tension loads are approximately 2500 kN and 1800 kN respectively. The high compression loads are taken by end bearing on the rock and for this reason the pile toes are plugged with concrete. The tension loads are taken by dead anchors drilled beyond the pile toes and grouted into bed rock.

The vertical pile is a composite pile with the upper section comprising an external sleeve grouted onto the inner pile after the inner pile has been driven to bed rock. The external pile is a prefabricated section with flanges for wave screen plank fixing. The use of the composite pile allowed the inner pile to be designed for the forces below the bed level, with the grouted sleeve above bed level providing the additional stiffness to carry the high moments induced by the wave screen. This arrangement also helped to eliminate alignment problems with the plank fixings.

The screen planking material is greenheart timber and the deck is concrete. Handrails and lights are provided on the deck to provide a public promenade and viewing area.

The cost of the breakwater, excluding pontoons and other marina items, is approximately £1.8 million, of which approximately 65% is the cost of the tubular steel piles.

The Contractor for the project was Dean and Dyball (Western) Ltd from Exeter.

Figure 10 shows the project nearing completion.

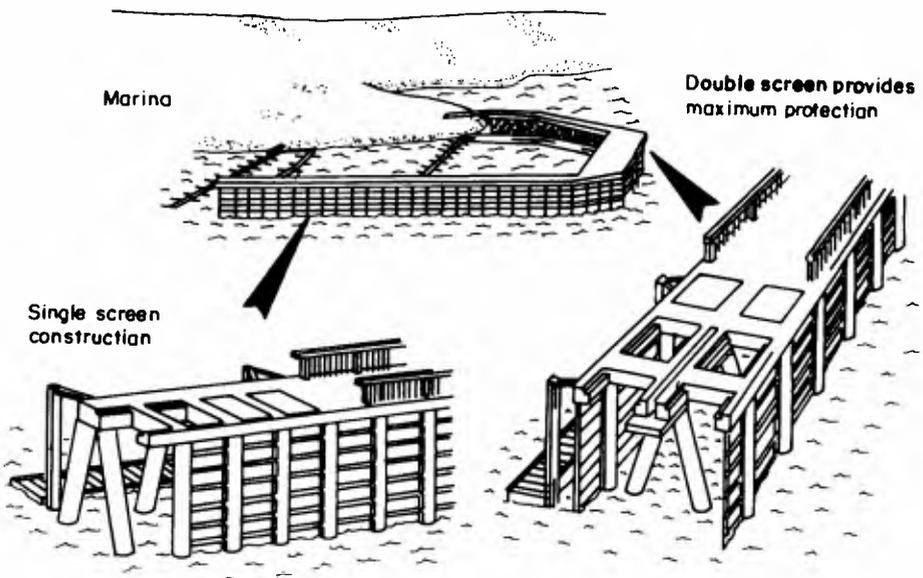


Figure 9: Typical views of single and double screen breakwater



Figure 10: Marina Nearing Completion

CONCLUSION

Based on visual information during the 4 month period since the breakwater was completed wave reflection into the shipping channels appears to be well within the permissible levels.

Without the use of a slotted vertical screen the marina project would not have been allowed to proceed, but using the relatively inexpensive methods of analysis and testing described in this paper a practical and aesthetically acceptable solution has been found.

ACKNOWLEDGEMENT

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