

## CHAPTER 120

### WAVE ATTENUATION AND CONCENTRATION ASSOCIATED WITH HARBOUR APPROACH CHANNELS

by

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#### ABSTRACT

Modern bulk carriers require deeper and wider harbour entrances. As a result long approach channels have to be dredged outside the breakwaters. These large channels cause considerable changes in the waves due to refraction and diffraction which effect the wave conditions in the harbour entrance.

As part of the Richards Bay harbour entrance studies, two phenomena were found to considerably effect the design of the entrance layout, namely:

- (i) wave attenuation for waves travelling near parallel to the channel axis, caused by refraction of wave energy away from the channel, and
- (ii) wave concentration on the channel side slopes, also mainly due to refraction, reaching a maximum for a critical approach direction of about  $25^{\circ}$  relative to the channel axis.

As a result of the wave attenuation, waves travelling within an angle of  $20^{\circ}$  either side of the channel axis were found to cause no problems whatsoever with regard to wave penetration into the harbour.

Unacceptable wave penetration was, however, experienced for wave directions close to the critical direction. A great number of variations to the entrance layout were tested to minimise this problem. The results showed the superiority of a flat channel slope of 1 in 100 above steeper side slopes and the beneficial effect of a Vee-shaped channel bottom. Also test results with irregular waves with regard to height, period and directions were found to be significantly different from those with uniform waves which considerably exaggerate the wave concentration phenomenon.

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INTRODUCTION

As a result of ever increasing ship sizes, greater channel depths and widths have to be allowed for in modern harbour entrance design.

Entrance channel depths of between 22 and 25 m are required for 150 000 dwt loaded ships whereas 250 000 to 300 000 dwt bulk carriers may require depths of between 25 and 28 m. Due to the limited depths which exist in most of the world's coastal areas, entrance channels become several kilometres long and, under these circumstances, it becomes impractical to protect the entrance channels by breakwaters over their entire length.

Moreover, safe manoeuvring of these huge vessels requires sufficiently wide entrance channels with widths varying between 300 and 600 m and preferably straight approach channels to avoid sudden course corrections in the channel.

As a result the conventional two-arm system of breakwaters is not very effective in reducing wave heights in harbours which incorporate these deep and wide entrance channels. Relatively short breakwaters sufficiently long to avoid excessive sediment deposition in the harbour entrance should therefore normally suffice.

This means that an approach channel of several kilometres long has to be dredged and maintained *outside* the protection of the breakwaters. During the studies for the design of the deep water port which is being constructed at present at Richards Bay in Natal (approximately 160 km north of Durban) it was found that the approach channel has a considerable influence on the wave conditions in the harbour entrance.

WAVE TRANSFORMATIONS EFFECTING HARBOUR ENTRANCE DESIGN

Wave directions and heights change due to wave refraction, diffraction, shoaling, bottom friction and percolation, as waves move into transitional and shallow water. Although bottom friction has been found to effect the wave heights and wave energy spectra in shallow water<sup>1,2</sup>, wave refraction and diffraction are by far the most important wave transformations in harbour entrance design. The effect on wave height due to shoaling only becomes important in very shallow water ( $d/L < 0,05$ , where  $d$  = water depth and  $L$  = deep water wave length) and the shoaling coefficient viz.  $k_s = H/H_0$  ( $H_0$  being the wave height in a water depth  $d$  and  $H_0$  in deep water), can readily be determined for a given wave length and water depth<sup>3</sup>. On the other hand, the effect of wave refraction and diffraction differ from case to case and can only be determined for a particular harbour layout by extensive hydraulic model testing or, in cases, with relatively simple boundary conditions, by calculation, i.e. mathematical modelling.

Conventional harbour layouts

Before the advent of the super class ships conventional harbours usually consisted of twin-armed breakwaters enclosing a sufficiently large area for mooring ships or protecting a river mouth or lagoon entrance in the case of inland harbours (see Figure 1).

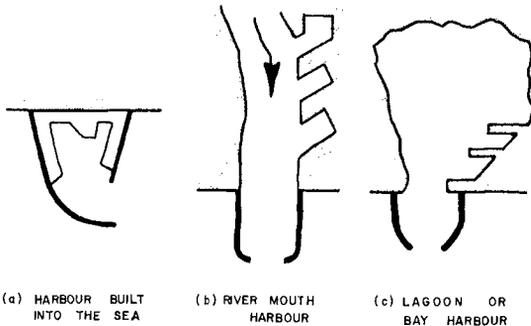


Fig. 1. Conventional type harbours.

Examples of the first type can be found at Kashima (Japan), Gordons Bay, Port Elizabeth, Cape Town (South Africa) and of the second type at Ijmuiden, Rotterdam (Holland), East London, Durban (South Africa).

The main purpose of the double breakwaters in the conventional harbour design was to reduce the wave heights in the harbour basin to acceptable limits and, in the case of sandy coasts, to protect the harbour entrance from excessive siltation.

To facilitate the reduction of wave heights the entrance widths were made relatively small, i.e. 50 to 200 m, which was acceptable in view of the high manoeuvrability of the smaller ships using these harbours.

Although in certain cases, for example Durban harbour, some dredging was necessary outside the breakwaters to maintain the required water depth in the harbour entrance the breakwaters usually extended to such a depth that no extensive outside dredging was necessary. For the design of these harbour entrances, including the breakwater structures, deep sea wave conditions could thus be converted to the harbour entrance area by standard refraction techniques and wave heights inside the harbour could be determined using diffraction diagrams<sup>3</sup> or on a hydraulic model.

Harbour entrances for super ships

The enormous increase in the size of super tankers has introduced a new dimension in harbour design. Where in the past the main emphasis was on providing sufficient entrance width between the breakwaters for ships to pass through, with the 17 to 22 m draught of the 150 000 to 300 000 dwt ships, deep and wide entrance channels have to be provided over considerable distances out to sea. Although sedimentation problems still play a role in the design of such channels<sup>4</sup> the channel alignment, width and depth are mainly determined by navigation requirements<sup>5</sup>. Preferably, the approach channel should be straight for easy manoeuvring and it has to be dredged out to sea to the point where the depth is equal to the required channel depth. Where the channel meets the shore relatively short breakwaters are provided to protect the channel against siltation caused by littoral drift and, to some extent, from wave penetration.

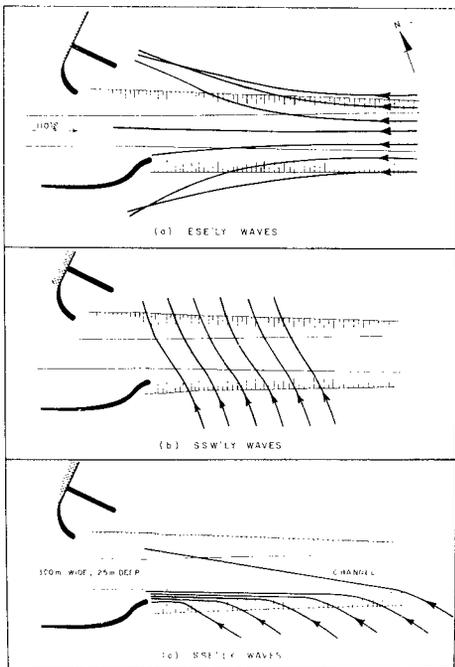


Fig. 2. Wave refraction caused by channel slopes.

The presence of a deep approach channel, however, considerably changes the local wave regime in the entrance area, mainly as a result of wave refraction caused by the channel.

As an example the basic layout of the Richards Bay harbour is shown schematically in Figure 2. This figure indicates for waves travelling in the direction of the channel a reduction in wave height in the entrance area near the breakwaters, Figure 2(a) for waves crossing the channel very little change, Figure 2(b) and for waves approaching the channel at a 'critical' oblique angle a considerable concentration of wave energy which may cause serious problems, Figure 2(c).

#### WAVE ATTENUATION

Based on conventional diffraction theory there would virtually be *no* reduction in wave heights inside a harbour with a large entrance channel as shown in Figure 2 for waves running in line with the channel axis. For instance for a 12 s period ESE'ly wave (i.e. almost parallel to the channel axis) the wave height inside the harbour would still be about 60 per cent of the incident height.

Tests on a 1 in 100 scale model showed considerably greater reduction in wave heights caused by refraction and diffraction in the approach channel. For the above wave condition the model gave a wave height in the harbour of only about 20 per cent which effectively solved the problem of the potentially difficult wave direction parallel to the channel axis!

#### Effect of dredged channels

The effects of a relatively deep dredged channel on the incident waves may be separated as follows:

- (i) A difference in wave celerity between the waves travelling in the channel and adjacent to the channel causes a considerable phase difference, resulting in wave diffraction on the channel edges;

- (ii) waves travelling on the channel slopes are refracted away from the channel.

Due to the difference in depth the part of the wave travelling in the channel will move faster than that in the shallower surrounding area. For example when the channel is 25 m deep and the surrounding sea bottom depth is 20 m the wave celerities for a 12 s period wave are 13,8 and 12,7 m/s respectively. Thus the wave in the channel would be  $180^\circ$  out of phase after it has travelled over 955 m in the channel (see Figure 3). Due to the difference in phase a mixed wave pattern

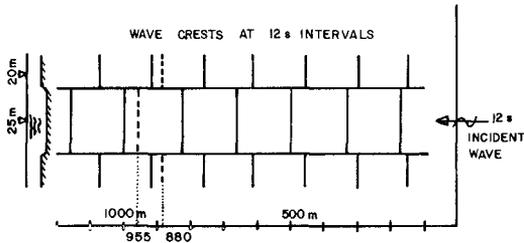


Fig. 3. Wave advance in dredged channel.

Preliminary tests in a wave basin, however, showed that with a phase difference of about  $90^\circ$  the waves break up into waves with individual crests inside and next to the channel and the average wave height at a distance of 700 m from the channel entrance was reduced by about 40 per cent for 12,0 s waves.

In practice the channel side slopes will not be vertical but will have a slope varying between 1 in 20 and 1 in 100. In addition to the diffraction effect, waves travelling on or near the side slopes will therefore refract away from the channel (see Figure 2(a)).

#### Richards Bay Harbour entrance

The proposed layout of the Richards Bay harbour entrance, which is designed for 150 000 dwt ships entering under virtually all conditions, is shown in Figure 4<sup>5,6</sup>. The layout includes a 3,5 km long, 300 to 400 m wide and 25 m deep dredged approach channel and a 1,2 km long main break-water.

Wave penetration studies formed an important part of the design of the Richards Bay harbour entrance. Tests were carried out on a 1 in 100 scale model to determine the wave heights in the harbour for all possible wave directions between SSW and ENE. As expected, the wave penetration for ESE'ly waves running parallel with the channel axis was greatly reduced due to diffraction and refraction in the entrance channel (see Figure 5) whereas normally this wave direction would have caused the worst wave penetration.

will result. Assuming vertical channel sides and constant water depth the resulting wave pattern could perhaps be approximated using conventional diffraction theory, i.e. the wave travelling in the channel will diffract out of the channel and the wave on the sides will diffract into the channel. The resulting waves are the sum of the two diffracted waves.

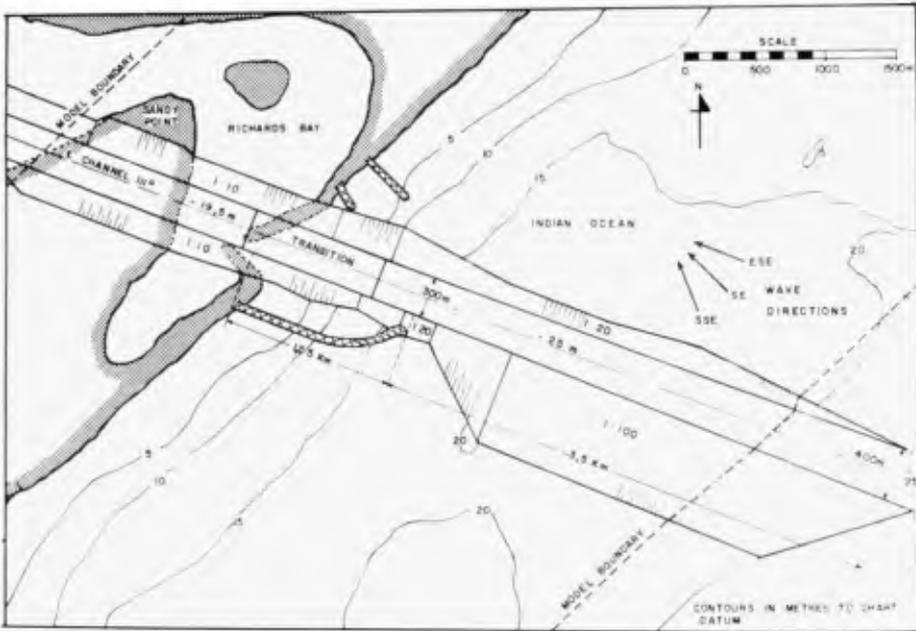


Fig. 4. Layout Richards Bay Harbour Entrance.



Fig. 5. Wave attenuation in entrance channel for ESE, 12 s waves.

Relative wave heights, as measured in the model for 12 s period ESE'y waves, are shown in Figure 6. For an incident wave height at the model boundary of 100 per cent, the wave height in the approach channel is reduced to about 50 per cent, further reducing to only about 15 per cent inside the harbour. High waves exceeding 130 per cent are seen to occur on the top of the 1 in 100 south channel slope.

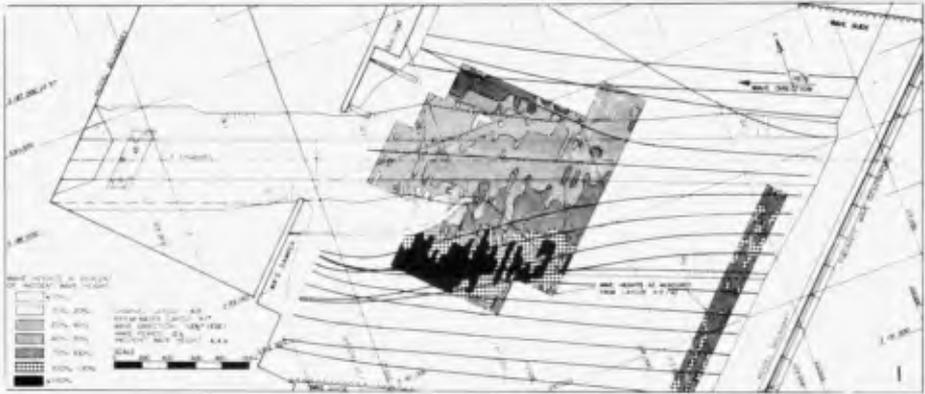


Fig. 6. Richards Bay, final layout, wave attenuation for  $112\frac{1}{2}^{\circ}$  (ESE) 12 s waves.

Wave refraction lines calculated by computer are super-imposed on the wave penetration diagram in Figure 6. The agreement, both qualitatively (wave pattern) as well as quantitatively (wave heights) is seen to be remarkably good.

#### WAVE CONCENTRATION

Wave penetration tests in the Richards Bay harbour model were initially done only for the wave directions E, ESE, SE, SSE and S ( $22\frac{1}{2}^{\circ}$  direction sectors) and wave penetration was found to be acceptable for all these directions<sup>6</sup>. At a later stage of the investigation, however, it was decided to test several directions between SE and SSE and it was found that unacceptable wave penetration occurred over a narrow direction sector of  $10^{\circ}$  to  $20^{\circ}$ , with a mean angle of approach relative to the channel axis of about  $25^{\circ}$  (or  $35^{\circ}$  relative to the corresponding deep sea wave direction).

This wave concentration caused a serious problem in the design of the Richards Bay harbour entrance layout. For the initially accepted layout<sup>6</sup> it caused wave heights considerably in excess of the prescribed 0,9 m maximum inside the harbour, it caused high waves in the harbour entrance creating problems to navigation and it resulted in very high waves on the south breakwater head.

#### Wave concentration on Richards Bay south channel slope

The wave concentration problem can be explained by *refraction theory*. As is shown in Figure 2(c), there is a 'critical angle' of approach when the waves will refract just enough to remain on the channel slope causing a large increase in wave height until a state of instability is reached and the waves move sideways across the channel. This can be seen clearly from Figure 7 which also shows the large areas where the wave heights exceed the incident heights by more than 30 per cent (the layout used for this test had a slope on the south side of the channel of 1 in 20 compared with 1 in 100 shown in Figure 4).



Fig. 7. Relative wave heights in entrance channel without breakwaters.

Again for a channel depth of 25 m and a depth of 20 m for the surrounding area the 'critical' refraction angle, which is the angle of wave approach resulting in a refracted direction parallel to the channel axis for a 12 s wave, is found as follows:

$$\sin \alpha = \frac{C_{si}}{C_{ch}} = \frac{12,7}{13,8} = 0,92 \quad \text{or} \quad \alpha = 67^\circ$$

where:  $\alpha$  = angle between normal to orthogonal and channel axis

$C_{si}$  = wave celerity to the side of the channel

$C_{ch}$  = wave celerity in the channel

Thus the critical angle of wave approach is  $23^\circ$  or with the  $111^\circ$  channel direction the critical direction of wave approach at the channel entrance becomes  $134^\circ$ , which corresponds to a deep sea wave direction of about  $141^\circ$  (between SE and SSE). This agrees very closely with the average critical direction found from the various model tests on the Richards Bay model which was found to be about  $25^\circ$  (see Figure 8).



Fig. 8: Wave concentration on channel slope for SSE, 12 s waves.

Although the wave concentration caused by the Richards Bay south channel slope can thus be explained effectively by refraction theory it is clear that wave heights will reach a limiting value where the assumption of *no* transverse energy transfer does not hold any more. Particularly for a relatively flat channel

slope, in the initial wave height build up, diffraction will play a minor role only but as the waves get higher and the difference in water depths becomes greater near the shore, diffraction becomes more important.

As may be seen from Figure 7 a point of instability is reached where the high waves leave the channel slope. In this particular case the maximum wave height on the channel slope before it crossed into the channel was 1,6 times the incident wave height.

*Similar problems can be anticipated where long entrance channels have to be dredged outside the breakwaters.*

Examples may be found at Le Havre with its 12,5 km long 300 m wide entrance channel and at Europort with its 12 km long by 600 to 400 m wide channel. In the latter case, unexpected damage occurred to the north breakwater which is probably caused by similar wave concentration on the channel side slope.

#### Methods to reduce wave concentration

Although the wave concentration occurs only for a small wave direction sector at Richards Bay, this sector is in the dominant wave direction and ways to reduce the concentration had to be explored. Theoretical studies as well as extensive model testing were carried out using different channel slopes, widths, degree of channel tapering (widening towards the seaward end), breakwater shapes and channel bottom configurations<sup>7</sup>.

Standard wave penetration tests were carried out for various layouts for all the wave directions in the critical range between SE and SSE. The effects of the changes were judged from relative wave height diagrams and from the average wave height inside the harbour opposite Sandy Point (see Figure 4).

The effect of *channel slope* was investigated by changing the south channel slope, which was responsible for the wave concentration, from the maximum stable slope of 1 in 20 to 1 in 30, 1 in 60, 1 in 80 and 1 in 100 respectively. Various composite slopes were also tested. Some of the test results are shown in Figures 9 to 12.

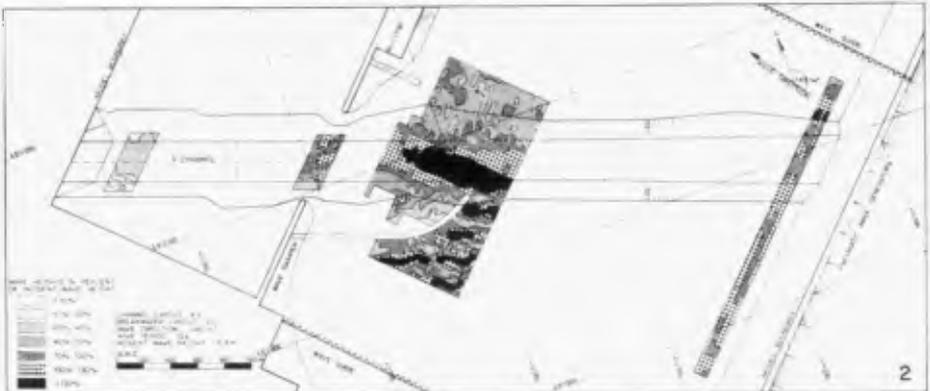


Fig. 9. Wave penetration diagram, 1:20 slope,  $146\frac{1}{4}^{\circ}$ , 12 s waves.



Fig. 10. Wave penetration diagram, 1:60 slope,  $146\frac{0}{4}$ , 12 s waves.



Fig. 11. Wave penetration diagram, 1:100 slope,  $146\frac{0}{4}$ , 12 s waves

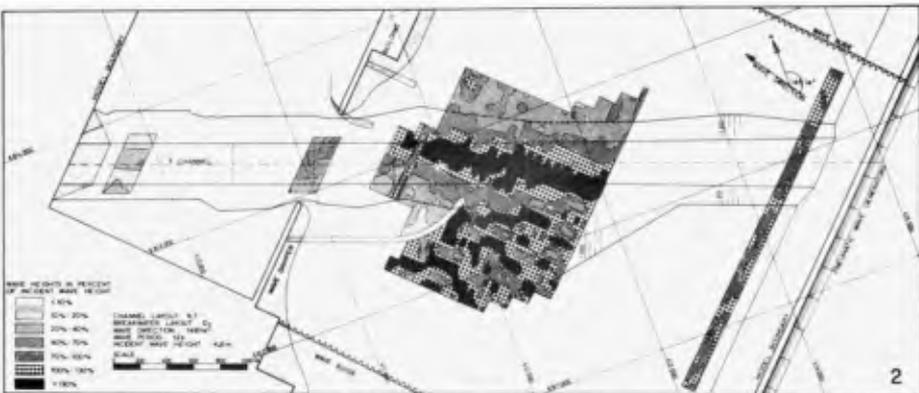


Fig. 12. Wave penetration diagram, 1:20/1:100 Composite slope,  $146\frac{0}{4}$ , 12 s waves.

As may be expected, the wave concentration spreads out as the slope decreases but only for a 1 in 100 slope was a satisfactory reduction obtained in the wave heights in the entrance channel (see Figure 11). To reduce dredging costs, composite slopes were also tried but were found to be ineffective, as can be seen from the example in Figure 12.

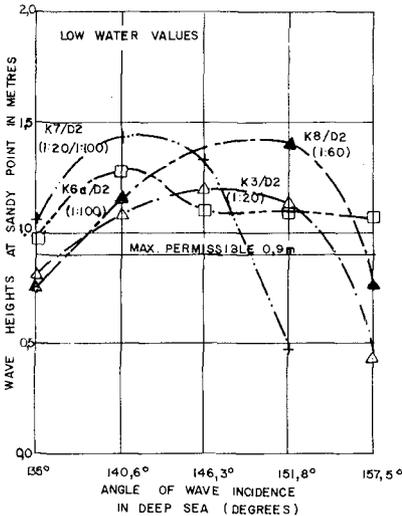


Fig. 13. Wave penetration at Sandy Point for various channel layouts.

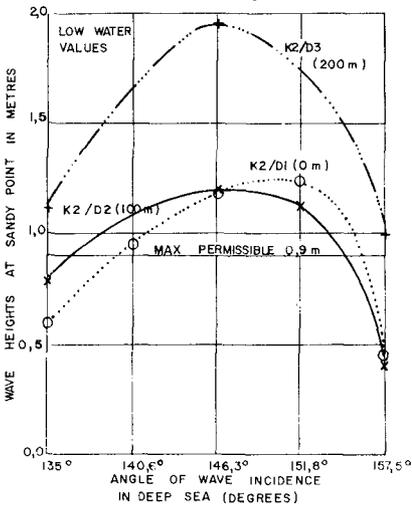


Fig. 14. Wave penetration at Sandy Point for various distances of breakwater tip to channel edge.

The wave heights inside the harbour are shown in Figure 13. The change of slope from 1 in 20 to 1 in 100, involving  $4 \times 10^6 \text{ m}^3$  extra dredging, is not too significant whereas the 1 in 60 and the composite slopes are considerably worse. The wave heights in the entrance channel, however, were much lower with the 1 in 100 slope (see Figures 9 and 11) and in later tests with different breakwater configurations it was found that the 1 in 100 slope was also superior with regard to wave penetration at Sandy Point.

The 1 in 100 south channel slope was therefore accepted as the best solution for the Richards Bay channel layout.

The effect of the distance of the breakwater tip to the channel side (bottom side) was also studied. The basic layout used was the same as shown in Figure 9 and wave penetration for distances of 0, 100 and 200 m respectively are shown in Figure 14. From the navigation point of view the breakwater tip should be as far as possible away from the channel side and the optimum distance is thus 100 m because wave penetration was found to increase significantly for greater values.

The effect of channel tapering was also tested. The increase in channel width from 300 m to 400 m shown in Figure 4 at the seaward end of the channel is required to compensate for the reduction in accuracy of navigational aids for greater distances offshore. Although no improvement was expected, a channel layout tapering from 300 m to 700 m was tested, but the results showed

a threefold increase in the frequency of occurrence of wave heights in excess of the maximum possible 0,9 m inside the harbour. This increase is due to additional wave energy converging towards the entrance area and the entrance channel should therefore have the minimum acceptable widening towards its deep end.

Breakwaters 1,2, 1,7 and 2,2 km long respectively were tested to study the effect on wave concentration of *breakwater length*. For the particular approach channel length at Richards Bay (about 4,5 km total) a 2,2 km breakwater length was found to be needed to avoid excessive wave build up on the channel slope. Although a long breakwater will thus provide a solution the high cost will in most cases be prohibitive.

Different south breakwater *shapes* were tested in a further attempt to reduce wave penetrations due to concentration on the channel slope. It was found that conditions inside the harbour improved significantly for a breakwater with a short section parallel to the channel axis, as can be seen from a comparison of Figures 11 and 15. Larger sections of the breakwater parallel to the channel were not acceptable because of considerable reflection of E'ly waves into the channel and the 'short kink' breakwater shown in Figure 15 was concluded to be the best layout.

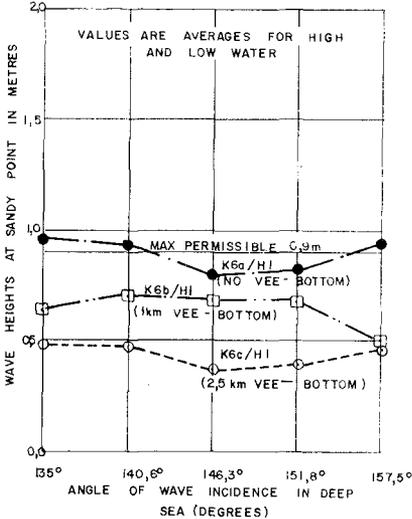


Fig. 15. Wave penetration diagram, short kink breakwater,  $146\frac{1}{2}^{\circ}$ , 12 s waves.

A very effective way to reduce the wave heights in the entrance channel, was found to be the provision of a *Vee-bottom* in the channel. The effect of the Vee-bottom is to refract the wave energy towards the channel sides where it moves out of the channel, or is dissipated against the sides. Two different conditions were tested using the basic layout shown in Figure 4, namely:

- (a) A Vee-bottom was provided over the approximately 1 km transition section from -25 m to -19,5 m channel depth (centre line depth was made -25 m and at the channel edges -20 m);
- (b) the above Vee-bottom was extended into the inner channel over a further 1,5 km (centre line depth -25 m, edges -18,5 m).

The test results in Figure 16 show a considerable reduction in wave penetration, even for the 1 km Vee-bottom which involves only  $0,5 \times 10^6 \text{ m}^3$  extra dredging. Although there are obvious practical disadvantages, this solution is very effective and in the case of wide entrance channels it is probably the only solution to the wave penetration problem.



*In certain cases a Vee-bottom can also be provided in the outer part of the channel to reduce the wave heights in the harbour entrance area. In fact, in the case of a harbour entrance for a marina at Muizenberg near Cape Town one proposed solution involved the provision of a specially shaped offshore depression combined with relatively short breakwaters which was found to reduce wave penetration to acceptable limits and, at the same time, improved entry conditions<sup>8</sup>*

Fig. 16. Wave penetration at Sandy Point for various Vee-bottom channel beds.

#### Wave penetration Richards Bay proposed layout

The final layout for the Richards Bay harbour entrance arrived at after some two and a half years research into the wave concentration problem is shown in Figure 4. Relative wave heights for ESE'ly waves are shown in Figure 6, which shows the considerable wave attenuation for the direction parallel to the channel, and for the 'critical' direction of  $146\frac{1}{2}^\circ$  in Figure 17, which shows the remaining wave concentration on the south channel slope.

Figure 18 gives the SE'ly wave condition, which results in far the worst wave attack on the South breakwater, causing serious problems to the stability of breakwater armouring. Figure 19 gives the 'critical' wave direction of  $77^\circ$  for the north slope of the channel ( $77^\circ$  deep sea or  $90^\circ$  at the seaward end of the channel). Although some concentration is noted on the north channel slope this causes no problems whatsoever.

Wave *orthogonals*, based on conventional refraction theory, are superimposed on the wave penetration diagrams of Figures 17, 18 and 19. The *agreement* between model tests and refraction theory is again seen to be very good, both qualitatively as well as quantitatively.



Fig. 17. Richards Bay final layout wave penetration for  $146\frac{1}{4}^{\circ}$ , 12 s waves.

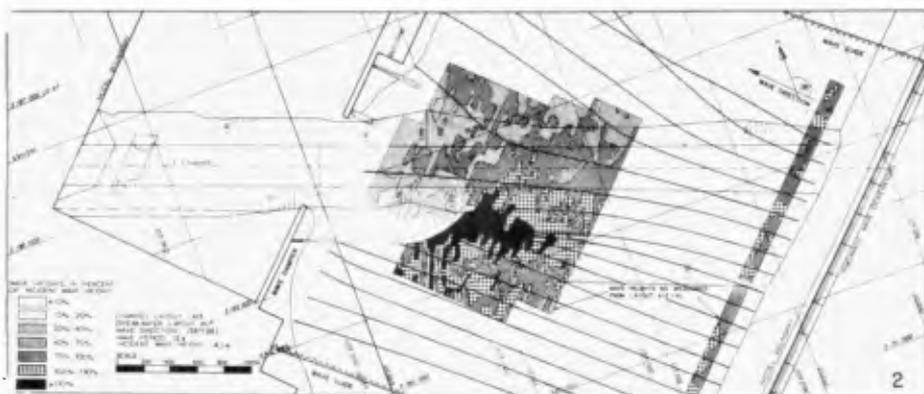


Fig. 18. Richards Bay final layout, wave penetration for  $135^{\circ}$  (SE)

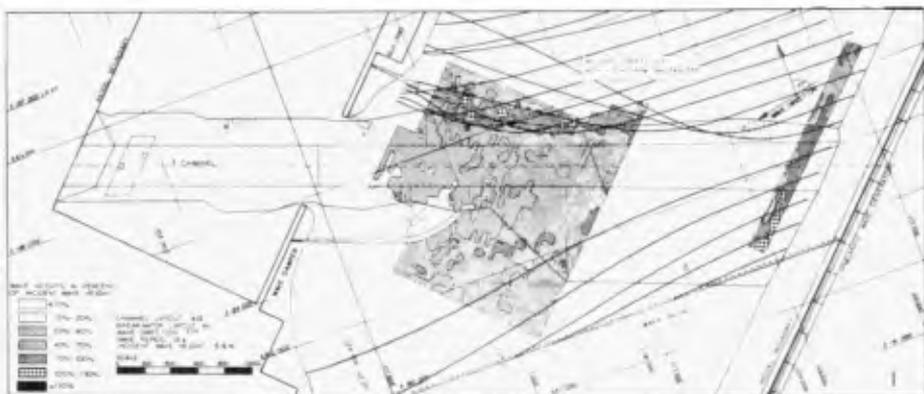


Fig. 19. Richards Bay final layout, wave penetration for  $77^{\circ}$ , 12 s waves.

The wave penetration inside the harbour opposite Sandy Point is shown in Figure 20 for 12 s waves and all directions tested. Wave conditions inside the harbour were not to exceed 0,9 m more often than once in 10 years. As may be seen from Figure 20 this requirement, however, was not met for the layout shown in Figure 4. The only way to further reduce the wave penetration was found to be a 1 km long Vee-bottom (see Figure 16).

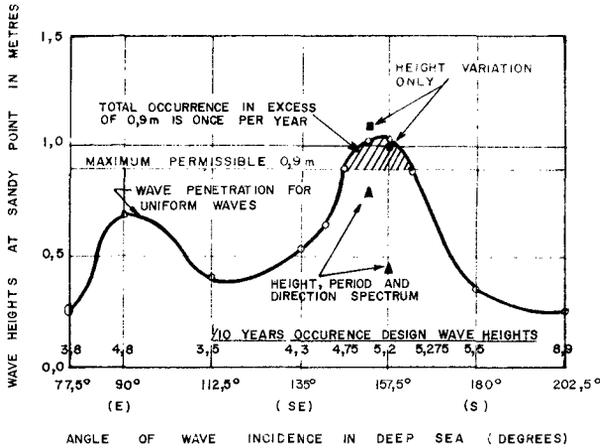


Fig. 20. Wave penetration at Sandy Point for Richards Bay final layout.

#### EFFECT OF IRREGULAR WAVES

The above considerations are all based on uniform waves which seldom occur in nature. Since the wave concentration only occurred in a narrow direction band it was decided to investigate the effect of irregular wave spectra particularly of a *direction spectrum*.

With the available wave generating equipment it was possible to generate pseudo-random waves both with varying wave heights only as well as with simultaneous direction, height and period variations. Tests were carried out with the basic wave directions of  $152^{\circ}$  and  $157\frac{1}{2}^{\circ}$ , which are in the critical direction sector, and a dominant wave period of 12 s. The direction spread achieved was  $7^{\circ}$  (model input) which can be accepted to occur under most conditions found in nature<sup>9</sup>.

Obviously, the wave measuring method used for the uniform waves, which was done for 10 s intervals on a close grid of 0,35 m by 1 m, could not be used for the irregular wave tests. Instead, the incident waves and the waves inside the harbour were measured simultaneously at six points for a 10 minute period. Simultaneously recorded 1 minute sections of the records were spectrally analysed on a Varian 620.L.100 computer and the average ratio of Sandy Point and model input  $H_{mo}^*$  values

$H_{mo}^* = 4 \sigma \approx H_s$ , where  $\sigma$  is the root of the variance and  $H_s$  the significant wave height.

were calculated to determine the wave penetration. The results, shown as point values in Figure 20 show very close agreement between uniform wave results and the wave height spectrum only. However, the wave height, period and direction spectrum cases show a considerable reduction in wave penetration, particularly for the  $157\frac{1}{2}^{\circ}$  case, for which the reduction is more than 50 per cent! Thus, provided it can be confirmed that for the critical wave directions at Richards Bay the minimum wave direction spread is at least  $7^{\circ}$  the entrance layout shown in Figure 4 is completely satisfactory.

The above results prove that the wave concentration problem is, in accordance with refraction theory, caused by a critical wave direction which focusses the wave energy onto the channel slope.

*It also follows that the tests with uniform uni-directional waves greatly exaggerate the wave concentrating and that, for situations similar to the Richards Bay one, it will be necessary to tests with a spectrum of wave heights, periods and directions.*

#### CONCLUSIONS

Modern bulk carriers require wide and deep harbour entrances which could result in serious wave penetration problems for waves running near parallel to the entrance channel axis. However, due to the existence of a 3,5 km long approach channel at Richards Bay this problem was completely eliminated because these waves were considerably reduced in height due to wave transformation in the approach channel. In the case of Richards Bay, this applied to all wave directions within about  $20^{\circ}$  either side of the direction of the channel axis.

Since the avoidance of beam waves also improves the navigability of the entrance <sup>10</sup>it thus follows that the *direction of the entrance channel* should preferably coincide with the *dominant wave directions* to minimise wave penetration.

Particularly, when the incident waves are limited to a relatively small direction sector *selective offshore dredging* can thus be utilised to reduce wave heights in a harbour entrance considerably reducing the problem of wave penetration.

At Richards Bay the channel alignment had to be made  $111^{\circ}$  because of the existence of a sub-bottom gorge in the base rock and also to limit the length of the approach channel, which did not agree with the dominant wave approach of  $158^{\circ}$ . In any case, recorded wave directions covered a direction sector of about  $80^{\circ}$  (in the channel area) which is much larger than the above  $2 \times 20^{\circ} = 40^{\circ}$  sector. For the angles of wave approach in excess of  $20^{\circ}$  relative to the channel axis, serious *wave concentration* was found to occur on the *channel slopes* particularly when the angle was close to the 'critical' value.

From a comparison of model test results and wave orthogonals based on refraction theory, it was established that the wave concentration is mainly caused by *wave refraction* on the channel slope.

Based on extensive model tests it was found that this problem could be minimised for the Richards Bay harbour layout by providing a flat l in 100 south channel slope, by using a short kink at the end of the south breakwater and by dredging a Vee-shaped channel bottom. The optimum distance from the breakwater foot to the channel edge was found

to be 100 m and the channel should have a minimum widening towards its seaward end. The wave concentration of course, can be avoided by lengthening the breakwaters but in most cases this would be uneconomical.

As a result of the wave concentration on the channel slopes, special attention had to be given to the *stability of the breakwaters* which experienced wave heights greatly in excess of the incident wave heights used in the design.

It was found recently that the wave concentration and resulting wave penetration were considerably reduced when a wave height, period and *direction spectrum* was introduced in the Richards Bay model. It is thus clear that, in similar studies, not only should a large number of wave directions, possibly with only  $5^{\circ}$  in between, be used in the tests but *preferably irregular waves with regard to wave height period and direction should be used to avoid considerable errors in the results obtained from tests with uniform waves.*

*Measurements* of prototype directional spectra thus becomes a prerequisite for accurate reproduction in the model of prototype conditions. Moreover, model wave machines which can cope with directional wave spectra should be employed for these studies.

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