Hydraulics Research
Wallingford

SHORE PROTECTION BY OFFSHORE BREAKWATERS

A H Brampton Ph.D

J V Smallman Ph.D

Report No SR 8
July 1985

**Registered Office: Hydraulics Research Limited,
Wallingford, Oxfordshire OX10 8BA.
Telephone: 0491 35381. Telex: 848552**

This report describes work carried out within the research programme Commission B for the Ministry of Agriculture, Fisheries and Food. The study was carried out in the Coastal Processes Section of the Maritime Engineering Department of Hydraulics Research.

The departmental nominated officer is Mr A Allison. The Company's nominated project officer is Dr S W Huntington.

ABSTRACT

This report reviews the information available for the design and use of offshore breakwaters in shore protection. As an introduction to the subject the physical processes occurring in the lee of an offshore breakwater are described with reference to natural examples. This is followed by a survey of case histories, and mathematical and physical modelling techniques for offshore breakwaters. Some of the methods which are available for the design of a breakwater system are reviewed. Possible future developments in the design process are described, and the areas in which further research on the effects of offshore breakwaters is required are highlighted.

CONTENTS

EXECUTIVE SUMMARY

1. These paragraphs summarise a report on the role of offshore breakwaters in shore protection. The study was undertaken for MAFF - Commission B - Marine Flood Protection, during 1984 and 1985.

2. The objective of the report is to evaluate the effectiveness of offshore breakwaters on reducing wave action at the main line of sea defence works. This evaluation also includes an assessment of the potential of such structures in encouraging improvement of natural beaches.

3. As an introduction to the subject, the physical processes occurring in the nearshore zone in the lee of an offshore breakwater are described with reference to natural examples.

4. A major part of the study comprises a literature review of case histories of offshore breakwaters. It was found that whilst there is much experience of design and building these structures in areas of low tidal range, for example in Italy and Japan, less information is available on sites where the tidal range is appreciable. It is in such areas, in particular in the UK, where continual monitoring and appraisal of the offshore breakwaters and their effects would provide useful guidelines for future design.

5. The report also describes the mathematical and physical model techniques which may be used to assist in the design of offshore breakwaters. At present mathematical models may be used to predict trends in beach plan shape due to changes in nearshore wave climate, and also to optimise the layout of an offshore breakwater system, taking into account both diffraction and overtopping. Physical models may be employed both to assess the hydraulic performance of an offshore breakwater, and to represent the effects of an offshore breakwater system on an adjacent beach.

6. In addition the report suggests areas where further research into modelling techniques would be valuable.

1 INTRODUCTION

This report, commissioned by the Ministry of Agriculture, Fisheries and Food, assesses the role of offshore breakwaters and similar structures in sea defence applications around the UK coastline. Although such structures can be used on their own to protect a coast, it has been assumed in this study that their principal role will be to reduce wave action, and its harmful effects, at an existing coast line. Wave heights are reduced both directly, by reflecting or absorbing the wave energy incident on the structure, and indirectly by improving the beach within the area sheltered by the structure.

As will be explained later, it is comparatively straightforward to calculate some of the benefits provided by, say, a detached surface piercing breakwater. In contrast other aspects, including the prediction of the changes in the beach, both in profile and plan shape, present much more difficult and challenging tasks.

In Chapter 2 the general background of the development of artificial breakwaters is discussed. This includes examples of naturally formed coastal formations which demonstrate the effects of such structures on a beach, as well as a description of the physical processes involved.

This is followed in Chapter 3, by a review of relevant literature and case histories, mainly based on overseas experience, but also including a preliminary appraisal of three offshore breakwaters recently constructed along the UK coastline (at Rhos-on-Sea and at either end of Leasowe Bay on the Wirral). This review gives some general guidance on the design of such structures, although most of the examples cited are on coasts with a very much smaller tidal range than is typical around the UK coast.

Following from these general guidelines, Chapter 4 deals with the methods available for designing both submerged and emerged (surface piercing) breakwaters. The chapter also reviews the role of both physical and mathematical modelling as aids to design. Although there is already a considerable amount of research relevant to such design techniques, many problems remain, especially in connection with submerged and permeable structures.

Possible future developments in the design of offshore breakwaters, and the research required, are discussed in Chapter 5. This includes discussion on topics such as the effects on a beach caused by a permanent alteration to the wave climate and the role of beach nourishment in combination with the construction of offshore breakwaters.

The final chapter summarises the conclusions and recommendations of the study.

2 OFFSHORE BREAKWATERS - THE PHYSICAL PROCESSES

2.1 Natural examples

Along many sandy coasts around the world, especially in the Mediterranean Sea, natural offshore islands and reefs have caused accumulations of beach material in their lee. Often the effect is simply to produce a local increase in the beach width resulting in a bulge or cusp in the sheltered area. However when an island is sufficiently close to a shoreline, in comparison to its length, the resulting beach plan shape can be much more spectacular.

In such circumstances, a narrow 'neck' of beach material can form between the shoreline and the island, and this feature is generally referred to as a 'tombolo'. The word is of Italian derivation and is used there to describe any low lying sandy area near the coast and would be perhaps more correctly translated into English as 'links'. In coastal engineering, however, the term is reserved for the specific feature described above.

Tombolos can be almost perfectly symmetric resembling a wine glass stem in plan shape, or distorted to some extent either by obliquely incident waves or by the shape of the offshore island. Two very impressive examples demonstrating these points can be found on the Pakistan coast and are shown in Figs 1 and 2. Occasionally more complicated shapes occur such as the double (or even treble) tombolo as illustrated in Fig 3 which shows a famous example at Orbetello on the west coast of Italy.

It is also worth noting how varied the size of tombolos can be. At one extreme it is simple to create a tombolo in the laboratory, or to find examples of similar size on the shores of lakes and ponds. Typical examples on open coasts range from tens to hundreds of metres in length, but the examples on the Pakistan coast are several kilometres long, which is exceptional. The formation of these two tombolos, however, has been assisted by rapidly changing land levels, due to local seismic activity.

There are also a surprising number of similar features around the northern and western coasts of the United Kingdom. A good example of a beach cusp behind an offshore reef is seen in Plate 1, which shows part of the Dorset coast. It is interesting to note the underwater extension of the cusp which resembles an 'underwater' tombolo. This appears to be a common feature in UK waters, presumably as a result of the large tidal range. Further examples of submerged or 'low-water' tombolos can be found at St Michael's Mount in Cornwall, and at Burgh Island in South Devon. Fully emerged or 'high-water' tombolos are less easy to find although there are good examples at Llanddwyn Island on Anglesey and at Langness on the Isle of Man. It also seems likely that both Hugh Town, Isles of Scilly and St Ives in Cornwall are partially built on such tombolos. Tombolos can also form on shingle beaches, and the Portland end of Chesil beach is a good example.

The distinctive plan shape of emerged tombolos can only persist, however, if there is no nett alongshore drift of beach material. In the presence of such a drift, the area sheltered by the island gradually fills from the updrift side; one half of the tombolo is thus obscured although the coast often retains its indented shape on the downdrift side. A good example of this situation is provided by the coast at Hartlepool (Fig 4). The downdrift bay often provides a natural site for a harbour, as in this case. Such bays have attracted considerable interest in recent years as coastal engineers have tried to artificially create similar shapes, either to reduce erosion or provide sheltered harbour sites. As a result, such features have been described by a plethora of terms including zeta, log-spiral, crenulate or headland-bays. This interest is well demonstrated by the large number of references reviewed in Chapter 3.

There is no doubt that both tombolos and headland bay beaches are usually very stable as well as aesthetically pleasing natural coastline formations. It is therefore not surprising that when faced with erosion problems, engineers in areas such as the Mediterranean Sea tried to emulate nature, and protect coasts with artificial offshore islands. This decision was undoubtedly influenced by a supply of suitable rock, and the relatively unsophisticated construction methods required. Design of such offshore breakwaters, however, was based on empirical criteria, expressing for example, the ratio of breakwater length to the distance offshore. Further, these structures were used as a universal panacea for coast erosion, whatever its cause, and often the effect achieved was less than ideal. The remainder of this chapter is therefore devoted to a description of the hydraulic processes which occur in the vicinity of a nearshore island or breakwater.

2.2 Physical processes

The response of a beach to an offshore island is brought about by the joint action of waves and the currents which they create as they break. To explain the processes involved, it is sufficient to consider the effect of an island on an initially straight coastline, with normally incident waves. In such a situation, the breaking waves produce a current system similar to that shown in Fig 5a.

It will be observed that close to the shoreline, currents are driven into the lee of the island from either side. These currents are created by two separate mechanisms, namely the oblique angle of the breaking waves and the wave height gradient. The former mechanism is well known and has been described by many authors (see, for example, Muir-Wood and Fleming, Ref 1). The generation of shore parallel currents by an alongshore variation of breaking wave heights, is a less widely publicised phenomenon. Briefly, however, breaking waves produce within the surf-zone an increase in the mean water level, known as set-up. Set-up increases with increasing breaker height, and so in a situation where there is a marked alongshore variation in breaker height, a current flows laterally as a consequence of the varying mean water level within the surf zone. In the case of an offshore island, this mechanism thus creates a flow into the shadow zone from either side.

Referring again to Fig 5a, it should be noted that an inward flow sometimes also occurs along the rear face of the island. This is created as waves, diffracted around each tip, break and decrease in height as they travel further into the sheltered zone. The mechanisms involved here are the same as along the coast, but since the water depth is greater the currents are much smaller. Nevertheless it is quite easy to demonstrate in physical models the four-cell system shown in Fig 5a.

Given these currents, and the agitation caused by breaking waves which brings sediment into suspension, it is not surprising that beach material is swept into the lee of the offshore island. In consequence there is almost invariably a broadening of the beach behind the structure as shown in Fig 5b.

Following the formation of such a beach cusp, a similar current pattern to that described previously will exist, although the currents are reduced in intensity. Indeed they may be too weak to move material any further seawards, and the beach may stabilise in this configuration. Such situations are not uncommon on the UK coastline, and a good example is provided by Carr House Sands, south of Hartlepool (see Fig 4). A variety of factors may explain the failure of such a beach 'cusp' to develop into a full tombolo, including the dimensions of the island compared to its offshore distance (both of which should be expressed as multiples of the length of the incident waves). In areas of large tidal range, such dimensions and lengths vary with water level, and the hydraulic processes are also affected by tidal currents.

It should also be noted that Fig 5b indicates some erosion of the shoreline on either side of the shadow zone. This erosion is explained by beach material being displaced laterally to form the cusp, but it is worth making the point that some shoreline recession may take place even if extra beach material is supplied artificially in combination with the construction of the offshore breakwater. The ultimate development of the shoreline, for the simple geometry being considered here, is a full tombolo as shown in Fig 5c. Here there are no currents, and in consequence the plan shape of the beach is in equilibrium (for the particular incident wave condition).

It should be noted that in this situation, wave crests strike the tombolo at an angle which would cause currents to flow 'outwards', ie away from the shadow zone towards the open coast on either side. This mechanism is exactly counter-balanced by the alongshore variation of wave heights, which tends to produce an 'inward' flowing current.

The balance between these two currents generating mechanisms (which in the first instance acted together to modify the beach plan shape) is described in a paper by Komar (Ref 2), although in the context of beach cusps rather than tombolos. As will be seen in the literature review, there has been further research into the theory of tombolo formation based on the ideas in that paper.

If the predominant wave direction produces an alongshore drift, the beach will develop rather differently than in the simple situations considered so far. This situation is illustrated in Fig 5d, and it should be noted that the current driving the sediment along the beach is diverted seawards within the shadow zone. As a consequence an offshore breakwater can function as an extremely efficient groyne. While this may be the intention, the consequential downdrift erosion is a potential danger.

So far this description of the interaction between an offshore island and a beach has concentrated on the changes in beach plan shape. However, there is usually a corresponding change in beach profiles, especially directly behind the breakwater where the greatly reduced wave climate results in steeper beach slopes. Accompanying this change in beach profiles, it is often reported that material is transported onshore, ie to the upper beach from the seabed below the low water mark. However, it is substantially more difficult to anticipate changes in beach profiles than in plan shape, and much more work is required before reliable predictive models become available.

3 LITERATURE SURVEY

3.1 Review of case histories

Much of the design of new offshore breakwaters in countries such as Japan, Italy and Israel, is based on experience gained from observing the performance of earlier structures. In this section of the report we will review case histories which detail the effects of various breakwater systems on the coastline.

A number of field studies of the effects of offshore breakwaters have been carried out in Japan. Toyoshimo (Ref 3) presents design criteria for detached breakwater systems which are based on their observed performance over a number of years. These design recommendations have been put into practise with some success in Japan. As an example, Toyoshimo reports on the effect of a system of three detached breakwaters off the Kaike coast, Japan (maximum tidal range less than 1m). In this area which had previously experienced serious coastal erosion problems the introduction of the offshore breakwaters led to sand returning to the coast and the subsequent formation of tombolos in the lee of the structures. Toyoshimo (Ref 4) subsequently reports on the addition of further breakwaters to the same stretch of coastline between 1974 and 1981 and examines the foreshore variation due to the complete breakwater system. He concludes that both shoreline and seabed have reached a stable position in contrast to the severe erosion observed prior to the construction of the breakwaters.

Many examples of the use of breakwaters in shore protection off the Italian coast are provided by Cortemegilia et al (Ref 5). In particular they examine the effect of man-made structures on the littoral zone, showing that the measures carried out often restore to equilibrium sites which were previously being seriously eroded. Amongst the areas quoted is Rivabella beach where, since the construction of a system of emerged detached breakwaters, tombolos have formed in the lee of the barriers to the extent that breakwaters are now joined to the coast thus increasing significantly the beach area. Cortemiglia et al also indicate some of the less favourable aspects of such a breakwater system. These include erosion of downdrift beaches, degradation in water quality due to limited water exchange and the creation of rip currents in the gaps between the breakwaters. A suggested alternative to emerged, ie surface piercing breakwaters which would overcome some of these unwelcome effects are submerged barriers. Details of physical model tests conducted by Aminti et al (Ref 6) to examine the use of submerged barriers for shore protection are given in section 3.2 below.

Fried (Ref 7) describes the history of the breakwater system which protects the Tel-Aviv coast in Israel. The Tel-Aviv coast is characterised by narrow sandy beaches; the seabed in the nearshore shallow water has a number of rock outcrops and further out in deeper water the seabed is sandy. It was found that whilst during the summer months (maximum tidal range 0.2m) a reasonably wide beach was maintained, the winter storms reduced the width of the beach to zero in certain areas. In order to prevent this erosion and at the same time increase the beach area, a series of five offshore breakwaters, either detached or shore connected were built. The scheme was successfully completed in 1968 with the foreshore having been transformed to a wide sandy beach. Tombolos formed in the lee of the breakwaters expanded steadily and reached an equilibrium state after three years. Fried reports that following this success it was proposed that further protected beaches should be created on the Tel-Aviv coast by similar means. A system of four detached breakwaters was designed, with the aid of a mobile bed physical model (further details of which are given in 3.2), and construction of these completed in 1976. The additional structures also proved to be successful in that the existing beach was substantially widened without causing any permanent erosion to the adjacent shore.

The case histories given above indicate the use of conventionally built breakwater systems in areas of low tidal range to protect and improve existing beaches. Evidence of the use of less conventional offshore structures, eg composed of tyres or sand-filled bags, for protection of inland or sheltered sites in the USA with varying degrees of success is presented in Low Cost Shore Protection (Ref 8).

Another novel means of coast protection is given by Zwamborn et al (Ref 9) who detail the mobile bed model tests, construction and subsequent monitoring of an underwater mound acting as a submerged breakwater in Durban, South Africa. The mound, composed of spoil from dredging operations in Durban harbour, was used to protect nearby beaches which had been gradually eroded over a number of years. Model tests were performed to design and position the mound so that it would remain stable and have a beneficial effect on the beaches in its lee. Monitoring of the completed underwater structure has shown it to be reasonably stable and successful in providing protection for Durban's shoreline.

Finally we turn our attention to offshore breakwaters recently constructed in the UK with the aim of providing coast protection. It should be restated here that most of the experience gained in designing and building offshore breakwaters is in areas whose tidal range is very much lower than that generally found in British coastal waters. The spectacular results in which wide sandy beaches and tombolos are created in countries such as Israel and Italy should not be anticipated in this country. Detached offshore breakwaters in the UK should be viewed as a means for reducing wave action, and its consequent detrimental effect on the coastline, in the lee of the barrier.

The use of offshore breakwaters around the British Isles for coast protection is a relatively recent development. Rubble mound breakwaters have been built at Rhos-on-Sea and Leasowe Bay, The Wirral, within the last five yeas and a system of detached and shore connected breakwaters at Kings Parade, New Brighton has recently been completed. As all the breakwaters mentioned are relatively new constructions, only a limited amount of literature on the design materials used and their subsequent effect on the shoreline is available. An assessment of their performance so far is largely based on visits to all three sites made during August 1984. It should be emphasised that it is too early to judge the long term response of the shoreline to the structures and only initial impressions of their effects are given.

The rubble mound breakwater at Rhos-on-Sea (maximum tidal range approximately 7m) was constructed with the intention of alleviating frequent flooding of the residential area behind the existing seawall (see Fig 6a). A detached offshore breakwater was used at this site in preference to an alternative measure requiring the height of the seawall to be increased to a level which was considered to be visually intrusive. Prior to construction of the breakwater flooding behind the seawall could occur on almost every occasion when gales between north and east coincided with high tide, the worst conditions arising with north-easterly gales (Owen, Ref 10). The breakwater is positioned at the low water mark facing east-north-east and was designed and constructed to prevent overtopping of the seawall, and consequent flooding, for a 3m wave on a 100 year water level. During construction of the breakwater it was noted that waves from a north-easterly direction were entering the gap at the northern end of the breakwater and running along the seawall. As this was thought to be undesirable, armour stone rejected as unsuitable for the breakwater was used to make a groyne north of the breakwater. Since completion of the breakwater and groyne, material has begun to accrete in the lee of the structures (see Plate 2). In particular it can be seen by comparing beach profiles from surveys done in February 1980, July 1984 and September 1986, Figs 6 and 7, that a reasonable quantity of sediment has accumulated in the lee of the breakwater and also a very large amount of shingle has accumulated in the lee of the groyne. The beach profiles show a characteristic steepening since the construction of the breakwater with beach levels at the toe of the seawall being increased by as much as 2m. It is also worth noting that material has also been deposited at the offshore toe of the breakwater.

The offshore breakwaters at Leasowe Bay which were completed in summer 1982 are shown in Plate 3. A full report of the historical development of Leasowe Bay and the investigations made before the breakwaters were designed and constructed is given in Barber and Davies (Ref 11). The wave climate in this area is characterised by a high tidal range (maximum tidal range approximately 8m) with strong tidal currents close to existing revetments. The system at Leasowe Bay consists of two rubble mound breakwaters which are emerged at all states of the tide (the eastern-most breakwater is shore connected) and a fishtail shaped groyne (see Plate 3). The scheme was devised to increase beach levels locally and to retard further erosion of the North Wirral coastline. Since construction has been completed beach levels in the lee of the structures have increased, particularly adjacent to the shore connection of the eastern-most breakwater and in the vicinity of the fishtail groyne. Observations made at the site indicate that there are still strong tidal currents between the two breakwaters. The stone aprons at the breakwater ends seem to have been successful in dissipating the energy of waves breaking over the roundhead and no local scour is apparent. Beach levels have also increased on the seaward side of the breakwaters.

The system of groynes and offshore breakwaters at Kings Parade, New Brighton, was completed in Spring

1985. The breakwater layout is shown in Fig 8. The breakwaters are designed to be submerged to a depth of 1m at mean high water. All of them consist of a sand core covered by a layer of bedstone which is armoured with specially designed precast reinforced concrete units, known as reef blocks, which stand on concrete pads cemented onto the underlying armour rock. The breakwater system at Kings Parade is intended to increase existing beach levels and to extend the shoreline to join the ends of the breakwaters at locations 1, 2 and 3 (see Fig 8). The location 1 and 3 breakwaters intersect the major wave trains and current directions, whilst the location 2 breakwater is to assist in moving the shoreline seawards. The breakwater at location 4 is a wave screen primarily intended as a coast protection measure.

In addition to the well known offshore breakwaters at Rhos-on-Sea and The Wirral there is a scheme underway at Dengie Flats, Essex, intended to prevent, or at least retard, further erosion of the salt flats in this area. The design for the breakwater at this site consists of placing a number of lighters offshore, separated by gaps of specified width, and sinking them. Before such a scheme was put into operation an investigation into its probable effectiveness was conducted by Pethick (Ref 12). A discussion of the methods used by Pethick is given in section 3.3. Work is underway at the site to place the lighters so it is too early to make any comments on their performance in preventing further erosion.

At all of the sites of offshore breakwaters in the UK that have been included here it is intended that the shoreline response should be monitored over a number of years. This should provide useful experience to aid with future design of such structures.

3.2 Physical model studies

Over the years many authors have reported the results of physical model tests designed to assess the performance of offshore breakwaters. Such physical model studies may be broadly divided into two areas. The first comprises model tests concerned with the measurement of the effect of particular breakwater characteristics (eg crest height, breakwater width, angle of face slope) on an approaching wave train. The second area includes experiments intended to observe the changes made to beach plan and profile by the modified wave climate resulting from the presence of detached offshore breakwaters.

In reviewing the effect of breakwater characteristics on an incident wave train one of the main points to be considered is the height of its crest relative to the still water level. A submerged breakwater, where the crest is at or below the still water level, affords protection to the shore area in its lee by attenuating the incident waves as they pass over the barrier. The submerged structure absorbs some of the wave energy by causing the waves to break prematurely. Some of the remaining energy is reflected and some transmitted shorewards. Submerged barriers are generally less expensive to construct and maintain than a conventional emerged breakwater whose crest may be above the highest expected tidal level. This is because the wave forces on a submerged structure will be less than those on a conventional structure as the waves do not break directly on the barrier. Therefore the submerged barrier offers a potentially economic solution in situations where complete protection from waves is not necessary or desirable. However in places with an appreciable tidal range a submerged breakwater may be relatively ineffective at high tide and simply act as a conventional emerged breakwater at low tide. Consequently it should be built to withstand the force of waves breaking directly on it. Therefore it is suggested by various authors (eg Johnson et al, Ref 13, Raman et al, Ref 14) that for maximum effectiveness the use of submerged barriers be restricted to areas of low tidal range.

Much research has been done in recent years with the object of investigating the performance and offering design guidelines for submerged barriers. Johnson, Fuchs and Morison (Ref 13) present a review of early work in the field together with results of their own regular wave tests where the waves were normally incident on submerged rectangular barriers. Their physical model tests concentrated on the effect of barrier height and width, relative depth and position of the breakwater on the transmission coefficient (transmitted wave height/incident wave height). The conclusions reached by Johnson et al as a result of their tests may be summarised as follows:

(i) A barrier of given relative height (barrier height/water depth) is more effective in damping (lower transmission coefficient) steep waves than flat waves. The effect of wave steepness is small when the crest of the barrier is above still water level.

(ii) For a given relative barrier dimension and wave steepness, the transmission coefficient is greatest for large relative depths.

(iii) A wide barrier (relative to incident wavelength) has a better damping effect than a narrow barrier especially for steeper waves.

(iv) The transmission coefficient is lower if the barrier is situated a wavelength or more seaward from the normal wave breaking point.

The wave damping effect of a submerged dyke was later considered by Nakamura, Shiraishi and Sasaki (Ref 15). They present the results of extensive physical model tests as a series of graphs (illustrating experimental trends) which are intended for practical use in determining the effect of submerged dykes on regular incident waves. From the graphs it can be seen whether the dyke will cause the incident wave to break, and also allow the transmission coefficient and changes in wave height and length for breaking and non-breaking waves to be determined.

Milne-Dick and Brebner (Ref 16) considered the modification made to the energy spectrum of an incident wave by both solid (thin and rectangular in cross section) and permeable (composed of horizontal nested tubes) submerged barriers. The experimental values of transmission coefficients for these rectangular barriers in infinitely deep water were in agreement with those calculated by Dean (Ref 17) provided the incident wave trough never falls below the breakwater crest. Milne-Dick and Brebner also found that for all the breakwaters they tested 36% - 64% of the energy transmitted was at a higher frequency than the incident wave. For a permeable breakwater their results indicated that there was a clearly defined minimum transmission coefficient whereas for an impermeable structure of the same size there was no such well defined minimum. This was thought to be due to breaking and turbulence eradicating fluctuations.

Dattatri, Raman and Jothi-Shankar (Ref 18) extended the work of Milne-Dick and Brebner (loc cit) to look at permeable and solid breakwaters of rectangular, triangular and trapezoidal section. Raman et al (Ref 14) found that for all the sections tested the effect of the breakwater position and dimensions were similar to those of Johnson et al given in (i) to (iii) above. In contrast to the results of Milne-Dick and Brebner, Raman et al found that there was very little difference in the behaviour of the transmission coefficients for permeable and impermeable breakwaters. However it should be noted that Dattatri et al used rubble mound breakwaters for their permeable tests whereas Milne-Dick and Brebner used nested tubes. Dattatri et al also recommend that the best design for an offshore breakwater has a sloping seaward face with a vertical backface, this is in agreement with experiments performed by Kabelac (Ref 19) on the sediment trapping ability of such structures.

Aminti, Lamberti and Liberatore (Ref 6) also present the results of a series of experiments concerned with breakwater shape and roughness. Their tests were carried out with the breakwater sections on both fixed and mobile beds with normally incident regular waves. The results for the mobile bed models appear to be inconclusive and the authors are continuing the test programme. For the fixed bed models the authors present a series of results and comment on the effect of the barriers on the waves, the velocity field near the barrier, set up and run up and the effect of barrier characteristics. They put forward design suggestions for limiting both offshore transport and scour at the landward toe whilst still maintaining reasonable water exchange in the lee of the breakwater.

In areas where the tidal range is appreciable it is more usual for offshore breakwaters to be constructed so their crest is above the highest expected stages of the tide. However, on both physical and economic grounds there will always exist the possibility of some incident waves overtopping the structure. As the height of the breakwater crest above the water level (freeboard) decreases the probability of overtopping increases. Several authors have conducted physical model tests in an effort to relate the rate of overtopping and the wave transmission coefficient (transmitted wave height/incident wave height) to the breakwater and incident wave characteristics. Goda, Takeda and Moriya (Ref 20) and Goda (Ref 21) give an empirical formula for predicting wave transmission, Seelig (Ref 22) compares these values with experimental results for a smooth impermeable barrier subjected to both regular and irregular waves. The formula due to Goda is found to provide a reasonable estimate for predicting the transmitted wave height in irregular waves. The results of Seelig's experiments also indicate that for emerged breakwaters the transmission coefficient increases as the incident wave height increases for a fixed breakwater configuration. The results for regular waves show that a high percentage of the transmitted wave energy is at a higher frequency than the incident wave and this is in line with the observations made by Milne-Dick and Brebner (Ref 16) for submerged barriers. In irregular waves changes in the spectral shape produced by the breakwaters were very small. Allsop (Ref 23) gives the results of a number of random wave tests conducted to measure the rate of wave overtopping and the coefficient of wave transmission. Allsop relates the wave overtopping to the dimensionless freeboard parameters, freeboard/significant wave height and freeboard/depth of water at toe structure. He concludes that the rate of overtopping is strongly dependent on the significant wave height but less so on the freeboard, whereas the waves transmitted by overtopping are dependent on the mean sea steepness.

The second area in which physical model tests have been used is in the investigation of the changes induced in beach plan and profile by the introduction of an offshore breakwater. Some of the physical modelling has been done to design a system of offshore breakwaters to meet the requirements of a specific site. For example, Fried (Ref 7) details the three dimensional moveable bed hydraulic model used to design the breakwaters off the coast of Tel-Aviv in Israel. The physical model was built in accordance with an extensive survey of the proposed site. This included preparing the model sand to match the distribution of grain size in the prototype. The model was calibrated with respect to the sedimentological processes which had been measured in the vicinity of two existing breakwaters further along the same stretch of coastline. In practice the time scale was found by comparing the rate of tombolo formulation in the model and for the existing breakwaters. Fried suggested that two time scales were necessary, one for the initial stage of tombolo formation, when the beach changes are due to shifting of local sand and longshore transport, and for the development of the tombolo due to onshore-offshore sand transport. Once these scales had been adopted a number of different designs were tested and the chosen design re-tested to gain an appreciation of sedimentological processes when the proposed construction schedule was correctly reproduced in the models. Fried concludes by comparing the implemented breakwater scheme with the model tests and finds that the actual performance of the breakwater system was in line with that predicted by the model. Sato and Tanaka (Ref 24) present the results of hydraulic model tests intended to find solutions to the problem of maintaining the artificial beaches at Ito and Suma in Osaka Bay, Japan. Recommendations based on the physical model studies are made for changes to the existing layout of offshore breakwaters and groynes in order to protect the beaches and promote water exchange at both sites.

A more general set of physical model studies to determine the beach changes due to a detached breakwater are presented in Rosen and Vajda (Ref 25). They review the work of a number of authors and give results from their own mobile bed physical models. The tests conducted were on breakwaters whose width and crest height were fixed but whose length and distance from the shoreline were variable. All the breakwaters tested were positioned parallel to the shoreline and subjected to normally incident waves of various different steepnesses (deep water wave height/ deep water wave length). Their results indicate that the significant parameters characterising the dimensions of a spit or tombolo are the relative length of the breakwater (compared to the distance from the original shoreline), the relative distance from the original shoreline (compared with the position of the breaker line) and the relative crest height (above mean sea level compared with incident wave height). Based on their results and other observations Rosen and Vajda propose and discuss a new explanation of when sedimentological and morphological equilibrium is reached.

Mimura, Shimizu and Horikawa (Ref 26) give the results of a series of model tests performed to study the influence of a detached breakwater on coastal change. The physical model results are also compared with those from a 'one-line' mathematical model (details of this will be given in section 3.3). As in the experiments conducted by Rosen and Vajda (loc cit) a mobile bed model was used, the waves being normally incident on a breakwater lying parallel to the coast. Mimura et al measured quantities relating to the wave and current fields, sand transport and bottom topography in order to assess the effect of a detached breakwater on waves, nearshore current, sediment transport and topographic change. They concluded that the positions of erosion and accretion appear along the main course of the current, the sand being transported by currents which developed shorewards of the breakwater. The eroded sand from along the shoreline far from the breakwater was transported into the shadow zone of the breakwater and the sand scoured from the sides of the breakwater was carried into the offshore region with the current.

It would seem from the literature reviewed in this section that there are certain areas in the physical modelling of offshore breakwaters which have received very little attention. In particular a series of model tests could usefully be conducted to investigate the effect of a breakwater on the beach plan and profile where the waves were not normally incident on the structure. Similarly no physical model tests appear to have been done, except for particular sites, on the effect of offshore breakwaters which are not positioned parallel to the original shoreline. Clearly there is scope for further physical model studies to improve our understanding of general processes involved in beach changes caused by the introduction of an offshore breakwater.

3.3 Mathematical model studies

The prediction of the long term evolution of a coastline due to changes in the nearshore wave climate is a complex mathematical problem. In recent years research has been carried out by a number of authors to produce mathematical models capable of predicting the response of a shoreline to the action of waves or the introduction of a man-made structure.

The simplest of the mathematical models is the so called 'one-line' method where the beach is represented by a single contour and the model deals only with changes in the plan shape. This idea was first put forward by Pelnard-Considere (Ref 27) and has subsequently been developed by many coastal engineers. Ozasa and Brampton (Ref 28) employ and extend the basic one-line method to model changes due to wave action in a beach backed by a sea wall. They include a new formula for alongshore transport of sediment which extends that developed by Komar and Inman (Ref 29) to situations where the wave height varies along the beach. (Normally alongshore sediment transport on a beach is caused by the oblique breaking waves but in cases where the waves are diffracted, for example by an offshore breakwater, the gradient of alongshore wave height must also be considered). Ozasa and Brampton also point out that an expression for the rate of onshore-offshore sediment transport may be included in the basic equations but in practise such a rate is difficult to quantify. The mathematical model results of Ozasa and Brampton are compared with those from a physical model study of a bay in Japan. The nearshore wave heights and directions for the mathematical model are taken from measurements made in the physical model, agreement between the mathematical and physical models was generally found to be quite good.

In the case of modelling a prototype beach the one-line model could still be used, the nearshore wave information being derived from a refraction model. In extending the method further to the case where an offshore breakwater was present, the input data would need to be obtained from a mathematical model of the nearshore zone, taking into account both refraction, diffraction and overtopping. This would clearly add further complexities to the problem. Mimura, Shimizu and Horikawa (Ref 26) employ the one-line method to model the effects of a detached breakwater on shoreline change. They use the formula for alongshore sediment transport rate developed by Ozasa and Brampton (loc cit) but with different values of the coefficients determining the balance between the forms due to oblique wave incidence and variation in breaking wave height. The new values of these coefficients were derived from Kraus, Harikai and Kubota (Ref 30). Again the input wave conditions to the mathematical model were taken from a physical model of the same breakwater layout. The results from the mathematical and physical models were compared but the two sets were found to disagree in the lee of the breakwater and in the shoreline region some distance from the breakwater. This disagreement was thought to be due to neglecting the cross-shore transport in the region away from the breakwater. In the lee of the breakwater the disparity in the results was thought to be due to assuming a constant value of the depth at which longshore-transport is thought to be negligible. Variation in the waves and currents around the detached breakwater will change the critical depth of sand movement and so invalidate the constant depth assumption. It is suggested by Mimura et al that further research is needed to develop a more rational model for assessing changes in bottom topography over a wide range of conditions.

The authors mentioned this far have only compared the one-line mathematical model with a corresponding physical model. Brampton, Franco and Noli (Ref 31) use the one-line method to predict future shoreline evolution and the effect of possible construction works on a stretch of coastline in Italy. The mathematical model is calibrated by using survey data gathered for the site between the River Arno and Viareggio harbour since 1934. The mathematical model was started with the observed shoreline conditions in 1934 and comparisons with other years survey data (including 1983) was found to be satisfactory. The model was then used, under certain assumptions, to predict shoreline charges up to the year 2034. Based on these results the authors make recommendations for coast protection works to prevent further coast erosion due to longshore drift.

Therefore it would seem that whilst a one-line method will not provide information on changing beach profiles, if carefully employed it can be useful in predicting evolution of the plan shape of a beach. Where the movement of sediment onshore-offshore is significant the two line theory of Bakker (Ref 32) provides a more sophisticated approach. The two-line method allows for the beach to be represented by two (non parallel) contours. The advantages afforded by the inclusion of onshore-offshore sediment transport and changes in beach slope may not be fully realised as such effects are difficult to quantify.

If still further information on the profile and plan of a beach are required then the n-line method of Perlin and Dean (Ref 33) could be considered. However although the method claims to give a reasonable approximation to bathymetric changes due to wave climate and the introduction of shore perpendicular constructions, Perlin and Dean point out several features which they feel could be improved. In particular, their method makes no provision for water level fluctuations in the nearshore zone. The longshore and onshore-offshore sediment transport are given by rather simple empirical formulae derived from laboratory tests. A number of adaptations would also be required to the basic model to include the effects of refraction and diffraction by offshore structures on the nearshore wave climate.

Finally, a mathematical model to investigate the efficiency of an offshore breakwater as a means of reducing wave erosion at a salt marsh site is given by Pethick (Ref 12), see also section 3.1. The proposed breakwater consisted of twenty lighters which would be floated into an optimum location for coast protection and then sunk. The aim of the research done by Pethick was to decide on the best position and spacing of the lighters to prevent further erosion of the maximum possible stretch of coastline. The wave conditions at the breakwater were calculated from deep-water wave conditions using a simplified model for predicting the changes in wave height and direction due to refraction. The refraction model used does not take into account energy losses due to the frictional effects of the shallow beach slopes or muddy seabed and it will overestimate the wave heights at the seaward side of the breakwater. The modifications made to the waves by a breakwater with several gaps are modelled using the work of Penny and Price (Ref 34), for diffraction, and Nakamura, Shiraishi and Sasaki (Ref 15), for overtopping. It should be noted that the diffraction theory of Penny and Price is for breakwater gaps which are 'wide' relative to the incident wavelength. Treating the gaps between the lighters as 'wide' will have led to the wave heights shorewards of the breakwater being overestimated. The combined effect of diffraction and overtopping at the breakwater is calculated by adding the diffraction and overtopping coefficients as suggested by Goda, Takeda and Moriya (Ref 20) and Treloar and Nagle (Ref 35). Pethick uses the results of laboratory tests to relate the wave heights at the coast to the erosion rates of the salt marshes. The results from the mathematical model for a number of breakwater gap spacings were verified against a physical model, the agreement between the two was found to be reasonable. Several different breakwater alignments were then tested using the mathematical model and the optimum position and spacing of the lighters was found. As stated in section 3.1, the placing of the lighters in the positions calculated using the mathematical model has been started but it is obviously too early to make any assessment of their performance.

From the literature it can be seen that although there are some very good mathematical models we are not yet at the stage of having a complete model for predicting the effect of an offshore breakwater on shoreline evolution. Certain aspects of the mathematical modelling, for example, calculation of overtopping coefficients and onshore-offshore sediment transport rates, could usefully receive more attention.

4 DESIGN METHODS FOR AN OFFSHORE BREAKWATER SYSTEM

This chapter discusses the various problems faced by the designer of an offshore breakwater system, and the methods which are available to help solve them. Probably the greatest of these problems are encountered at an early stage, namely deciding whether the use of one or more offshore breakwaters should be considered as a possible solution to a particular problem, and then how to draw up a first tentative plan. Unlike the design of a sea wall, or a groyne system, the engineer will not normally have the works either of his neighbours or his predecessors to guide his initial thoughts. The next section of the report therefore discusses these first steps.

Despite the lack of experience of offshore breakwaters in British waters, there, nevertheless, are a variety of methods which can be used to improve an initial design, both in cost and hydraulic efficiency. Indeed, as evidenced by the preceding chapter, there is possibly as much literature relevant to offshore breakwater design as far more conventional coastal defence methods such as beach nourishment or groyne systems. Section 4.2 briefly summarises the techniques by which the design of potential breakwater may be refined.

4.1 Developing the initial design

As with all coast protection methods, it is unlikely that a universal method for using offshore breakwaters will ever be developed. Each potential site is likely to have its own distinctive characteristics which will dictate the eventual solution adopted. Indeed in some circumstances the use of offshore breakwaters may not be appropriate. It is therefore worthwhile to first try to identify the types of problems that may be solved efficiently with one or more offshore breakwaters.

The immediate impact of such a structure will be to reduce wave heights in its lee, and to slow alongshore drift. These two effects themselves suggest applications for offshore breakwaters. If there is a localised, relatively short stretch of coast, or coastal defence, which is experiencing problems, an offshore breakwater may well offer an appropriate solution. For example, if a particular part of a sea wall is subject to overtopping or undermining, reducing the wave heights will bring an immediate improvement, without having to increase the sea wall height or extend its toe protection. Both of these latter options can be expensive and unpopular, especially on amenity beaches.

Similarly, problems often arise at the boundary between a sea wall and an unprotected stretch of coast. In such areas an offshore breakwater can at least spread erosion from over a wider frontage, and prevent the sea wall from being attacked. It is also possible that breakwaters may be deliberately designed to produce an efficient terminal groyne, perhaps in connection with a beach nourishment exercise. Such an application is, of course, fraught with potential danger and could have a dramatic effect on adjacent stretches of coastline if not carefully designed.

Because of the lack of experience of such structures in British waters, it is likely, in the immediate future, that the role of breakwaters may be largely limited to these localised types of problem. However, in other countries they have been used more extensively in order to change the nature of a whole stretch of coastline. An array of breakwaters built parallel to the shoreline can be used to completely arrest a nett alongshore drift of material and simultaneously reduce the intensity of the wave climate on that coast.

In view of the foregoing, the most likely applications for offshore breakwaters in the UK in the immediate future are when the alternative would be the rebuilding, raising or extending a sea wall over a stretch of a few hundred metres. It is also possible that offshore breakwaters may be considered either as terminal groynes to retain a beach nourishment, or in the place of conventional groynes where it is important to return a healthy beach, say in an area where tourism is of great commercial value.

Having identified a potential site for a breakwater installation, the engineer is then faced with deciding on the height, length (parallel to the shore), orientation and offshore distance of a suitable structure. Except for rather special situations, it is likely that the optimum orientation for a shore-protection breakwater will be parallel to the coastline. This will, in generally protect the greatest frontage of coast for a given structure length. In some cases it may be possible to place the breakwaters such that the dominant wave direction is taken into account. However, such a move should be given very careful consideration, as it may lead to the beach in the lee of the structures being exposed to occasional large storms from other than the dominant wave direction.

It is also likely, for the majority of applications, that the breakwater will be surface piercing at all states of the tide, if it is to perform satisfactorily. This general rule may perhaps be relaxed provided sufficient model testing is carried out, or if the object is simply to prevent scour at the base of a sea wall. The use of "submerged crest" breakwaters designed to improve a beach is mentioned again later in this section. For all other purposes, however, it is virtually essential for the breakwater to have an "emerged crest".

Having said that, it is not essential that the crest should be so high that it is never overtopped by waves. Most of the examples described in the worldwide literature, and those already built in the UK, are designed to allow some wave energy to pass over them in severe storms. Although this has some obvious impact on the design of both the crest and landward slope of the structure, the extra cost of providing a heavier construction there is outweighed by the much greater expense of extending the crest to a much higher, and potentially unsightly level.

The remaining dimensions of the breakwater (system) are the length of the structure, parallel to the shore, and its distance offshore. Although these dimensions are fundamental to a successful breakwater design, there are still no universally accepted methods for determining them. Clearly the distance offshore will be influenced by the likely structure height relative to the different tidal levels, and by the material and construction costs. General advice on the ratio of length and offshore distance is available, see for example Ref 48, where it is recommended that the breakwater should be further offshore than its own length to avoid the formation of a tombolo, which could have dramatic effects on the adjacent coast if there is any nett alongshore drift.

Conversely if a breakwater is placed too far offshore, then apart from the great construction cost, there is a danger that its sheltering effect will be 'diluted', ie, spaced over such a wide frontage by the effects of diffraction that its effect will be hardly noticeable.

An approximate method of calculating suitable dimensions is to assume that the structure will at least partially protect the coastline for approximately 2 to 3 times its own length, provided it is placed about 1.25 - 1.5 times its own length offshore. Thus if it is wished to protect a 250m stretch of frontage, the likely first design for a breakwater would be a structure approximately 100m long and about 125 to 150m offshore. Of course this is only a very simple and crude method of obtaining a first layout. The following section describes how this first idea may be improved.

Before that, however, it is worth discussing the possible use of submerged crest breakwaters. Clearly such structures have obvious advantages in cost over fully emerged breakwaters, and they have become popular in recent years in Italy. Here there has been a growing dissatisfaction with the visual impact of emerged breakwaters, and also the negligible tidal range often leads to the water between such breakwaters and the coast becoming virtually stagnant and heavily polluted. Submerged crest breakwaters overcome both these problems and also tend to eliminate the strong rip currents which sometimes occur between emerged breakwaters during even moderate wave activity.

However, their effect on the upper beach, in terms of improving coastal stability is rather less certain. Certainly intertidal outcrops of rock can often produce a locally wider beach, even in front of a sea wall. Despite this, there is no accepted method of designing an artificial structure to produce a similar effect. For the time being, therefore, the engineer must either rely on physical model tests, or be prepared to experiment, perhaps based on observation of natural features, such as shown in Plate 1.

4.2 Methods for improving breakwater design

Once an initial breakwater layout has been determined, there are a variety of mathematical and physical modelling techniques which can be used to improve that design, both in terms of construction costs and hydraulic performance. These techniques can be broadly divided into two areas, namely optimising the breakwater cross-section and modifying the plan shape of the breakwater layout. These two subject areas are now considered in turn.

The vast majority of offshore breakwaters that have been built as shore-protection structures around the world, have been constructed using randomly placed rock. It is likely that this popular form of construction will also be favoured in the UK especially on the western and northern coasts where land rock can be obtained from local sources. Indeed even on the eastern and southern coasts, imported rock from Scandinavia is already being considered and used as an alternative to concrete.

Apart from usually being the cheapest and simplest form of construction, rock mound breakwaters also offer hydraulic benefits, such as roughness and permeability which some alternative breakwater types lack. Rock breakwaters can be built either using land based plant, as in two of the UK examples already installed, or by using marine based plant, as at the recent harbour development at Ramsgate in Kent. This latter option may well have distinct advantages in areas where it is inconvenient to disrupt tourism or where access from the land is difficult.

Physical modelling to test the stability and performance of a breakwater has become a well accepted technique in the last decade or so, and is usually of great value to a designer. Not only can such testing avoid over or under-designing a structure, but such models also clarify some of the difficulties likely to be encountered during breakwater construction. It is worth making the point here that a considerable amount of experience has been accumulated from testing breakwaters over the last few years, and it is often possible to provide much useful advice to a designer on the basis of this experience. This may, in some cases, even obviate the need for specific model studies, although such studies are usually worthwhile at least in the latter stages of design.

Physical models can also be used to assess the impact of a structure on a beach, and to directly measure, say the reduction of overtopping of an existing seawall by the construction of a shore protection breakwater. Research has also been carried out which enables a designer to assess the durability of rock in the marine environment (Ref 36), and this can clearly be of great importance since offshore breakwaters have to endure in a particularly hostile area, preferably without the need for regular maintenance. The subject area of testing rock breakwaters is now rather large, and beyond the scope of this present report. For further, more detailed discussion and assistance the reader is referred to Powell and Allsop (Ref 37).

Despite the general prevalence of rock breakwaters, there are, however, some alternative construction techniques which should be considered. For example, a recent submerged crest breakwater near New Brighton, on the Wirral in Merseyside, has been built using concrete units designed to dissipate wave and tidal energy through turbulence created by the units. Work has also been carried out in Italy to develop another type of interlocking concrete unit also designed for use in a permeable offshore breakwater. This development has been initiated by pressure in Italy to stop the use of and quarrying for rock for aesthetic reasons.

It has also been suggested that breakwaters may be built using sand or other granular material, held in place by a layer of bituminised rock. This form of construction clearly requires little expensive material but will not be either as rough or as permeable as a rock mound. In consequence, the breakwater faces would probably need to be of a much flatter slope.

Finally, it is conceivable that an offshore breakwater could be built using concrete caissons. Although this would minimise the amount of material required to create such a structure, the highly reflective front face would be an obvious disadvantage on a granular beach. Such a structure would therefore require very careful consideration, and could only be recommended in special circumstances. For all the alternative types of structure, however, the use of physical models can be expected to be useful in improving the original, conceptual design.

Turning now to the optimisation of the plan layout of an offshore breakwater, it is again possible to use a physical model, and indeed this is often essential if several breakwaters are being proposed, or if the coastline is particularly complex. Such models allow the designer considerable versatility in testing different breakwater configurations. For example they can be used to look at the effects of different roundheads at the ends of the breakwater, or to investigate the value of a causeway or a similar structure linking the breakwater to the coast. In addition, a physical model with appropriately scaled bed material is at present the most reliable method of assessing the effects of a breakwater on the nearshore zone. There are still some difficulties with mobile bed modelling techniques, particularly with unwanted scale effects if the prototype bed material is sand. However, in exploring the response of the beach to varying tidal levels and changes in wave climate, caused by the introduction of an offshore breakwater, a mobile bed physical model is certainly a useful tool. In addition, such models are also valuable in allowing local people the opportunity to visualise the structures in relation to the existing coastline. This may be important since such breakwaters often cause concern regarding their visual and amenity aspects.

However, such physical model tests can be greatly assisted or even replaced by relatively cheap mathematical modelling methods. Admittedly the calculation of wave conditions behind a breakwater is not straightforward, since it is technically complex to model wave diffraction over a sloping sea bed, which itself is likely to adjust in shape. Nevertheless, such modelling can be usefully used to test the sensitivity of the performance of a breakwater to variations in length, offshore distance and even crest height. By combining such testing with parallel calculations of the variation in cost of the structure, it is possible to determine a configuration which gives good value for money. Detailed descriptions of some of the techniques available for this type of modelling may be found in References 38, 39, 40 and 41.

Mathematical models are also available to study the effect of a breakwater on the long term evolution of the coastline, but at present these models are restricted to rather coarse spatial and time resolution. Therefore the local changes in beach configuration, for example, in the lee of the breakwater, will be very poorly represented, but the overall response of the coast at some distance away will be accurately modelled. Further discussions on this issue may be found in Brampton, Franco and Noli (Ref 31).

Finally, in most situations the success of an offshore breakwater is likely to depend on its effect on beaches in its lee. If the installation of a breakwater halts or even reverses the trend of falling beach levels at the main line of defence works, or prevents further downdrift erosion which previously threatened to outflank these works, then it will probably be judged a success. It follows, therefore, that it would be valuable to evaluate and compare the effect of different breakwater arrangements on beach stability, in the long term, as part of the design process. This, unfortunately, is presently beyond the scope of mathematical modelling and indeed would be expensive to achieve even using large physical models, given the variability of tidal conditions, wave heights, periods and directions experienced around the UK coasts.

Experience has shown however, that offshore breakwaters invariably improve beach levels in their direct lee, and for a considerable distance on either side. In this sense they have been more successful than sea walls and indeed many groyne systems. The quantification of the benefit to beach levels, however, is a rather distant objective and is discussed in the following chapter.

5 FUTURE DEVELOPMENT

The aim of this chapter is to identify the extra information that is required by the designers of an offshore breakwater, and the future development work that is needed to provide that information.

Possibly the most important requirement is a distillation of the experience of designing and building such structures, together with an appraisal of the success achieved and any difficulties encountered. Although the existing literature, reviewed in Chapter 3, contains much useful information the number of articles concerned is quite large and they are variable in content and quality. Secondly there are only a small number of offshore breakwaters which have been installed in areas of high, or even moderate, tidal range and it is these structures which are likely to be of greatest interest to engineers in the UK.

At the time of writing this report, a survey of breakwaters in Italy is being carried out and will hopefully be available in late 1985. This survey will hopefully form part of a 'data base' for offshore breakwaters, which could be expanded to a much wider range of coastlines worldwide.

Such a data base should not only include details on design engineering, and coastal response, but also on, say, navigation, amenity, and ecological aspects. The recently built breakwaters on the Wirral, for example, have become a safe high-water roost for wading birds, whilst at Rhos-on-Sea local boat owners have used the calm water in the breakwater lee for mooring (and are now concerned by decreasing water depths as the beach accretes). In the Mediterranean, some breakwaters have been provided with a smooth cap to allow sunbathing or an offshore promenade. On the other hand, proposed breakwaters in the UK coast stimulated immediate objections from local fishermen and concern has also been expressed about the dangers that low crest breakwaters may pose to beach users.

Although there are only a few man-made structures around the UK coast that can be included in such a review, consideration should be given to the effect of natural offshore islands, reefs and banks which might give valuable supplementary data.

This appraisal may be particularly valuable in assessing the potential of submerged crest breakwaters which may be useful in improving beach levels in the lower part of the inter-tidal zone. Indeed, some prototype testing of such structures, both natural and man-made may well be a relatively cheap and effective method for investigating such structures.

Another area which has received scant attention is a breakwater's interaction with, and effects on tidal flows. This is given no consideration at all in Mediterranean countries, hardly surprisingly, and is an almost untouched subject area. In many areas around the UK coast, however, it is possible that benefits that would result from sheltering the coast from wave activity could be diminished by an unexpected increase in tidal flows behind the breakwater. This is a particularly important topic when a proposed site lies in or near an estuary or large tidal inlet.

Some further consideration also needs to be given to the specification of tidal level and current when calculating the effect of a breakwater on wave conditions. Although models for combining wave propagation with tidal currents are already in use, as are models for calculating wave diffraction around a breakwater, it is rather expensive to use these models too often.

Ideally, therefore, a few important tide levels, and incident wave combinations should be identified at an early stage to reduce the computational effort to an acceptable level. How these combinations can be chosen to give an indication of the overall performance of a breakwater is still an open question.

During the previous chapter, the use of physical models to assess the effectiveness of an offshore breakwater was discussed. Sufficient tests have now been carried out to give good advice on breakwater cross-section design although some further work is required on defining the character of waves overtopping a breakwater, or passing through it if it is permeable like most rock breakwaters. There is a much greater need for a series of tests to give guidance in plan shape layouts. These tests should not only be carried out with a mobile bed, but also on a rigid bed to give a better understanding of the basic hydrodynamics of an offshore breakwater. An investigation of this type is shortly to be started at Wallingford, in combination with the development of a parallel mathematical modelling study which will hopefully be able to extend specific model tests to more generalised situations.

Considerable research has already been undertaken into the methods of mathematically modelling offshore breakwaters, for example, as described in Ref 41. These models, however, are generally both difficult to understand and use in a coastal engineering context. Some further development is needed to make these models, or at least their results, simpler to apply.

Finally, there is no doubt that much fundamental work is required in predicting the response of a granular beach to a permanent alteration in wave climate. In the short term, valuable guidance could be gained from beach profiles taken, over a long timescale, before and after a breakwater has been constructed. Over a much longer time it may be possible to develop a theoretical framework to predict such changes, but this is a major task and it may take several decades to complete.

6 CONCLUSIONS AND RECOMMENDATIONS

(i) This report reviews the role of offshore breakwater in shore protection around the UK coastline. As an introduction to the subject the physical processes have been discussed and illustrated with reference to natural examples.

(ii) An extensive survey of the published literature on offshore breakwaters and their effects on the nearshore zone has been presented. This review covers case histories, and physical and mathematical model techniques. It was found that whilst there are a number of case histories for locations with a small tidal range, experience of offshore breakwaters in areas of high tidal range is very limited. The reported case histories were supplemented by observations and discussions with local engineering staff at offshore breakwater sites in the UK.

(iii) The situations in which offshore breakwaters could be a viable alternative to established coast protection methods have been reviewed. It was suggested that their use in the UK, in the near future, would mainly be for alleviating localised problems, rather than protect and develop long stretches of coastline as in, for example, Italy and Japan.

(iv) The amount of general guidance available for the design of plan layout for a system of offshore breakwaters in an area of appreciable tidal range is very limited. Some general design guidelines have been given in the report, but it is recommended that physical and mathematical model tests should be undertaken at the design stage to optimise the breakwater arrangement.

(v) Information on the design of a breakwater profile is far more readily available. However, it is still considered advisable to conduct physical model studies to test the stability and hydraulic performance of the design.

(vi) Existing mathematical and physical models can be used to improve the basic design of a breakwater system. However, both modelling techniques are still capable of being extended in order to give a better representation of the physical problem. In the long term research is required to investigate both the physical process of the response of granular beaches to a permanent change in wave climate, as caused by the introduction of an offshore breakwater, and to establish a theoretical framework for predicting long term changes.

(vii) The design of an offshore breakwater system in the UK could be aided considerably by appraisal of the performance of existing offshore breakwaters. In order to do this continual monitoring of both the structures and their environmental effects would be necessary.

7 ACKNOWLEDGEMENTS

The authors would like to thank Mr G B Norwood of the Borough of Colwyn, Mr C D Davis of the Metropolitan Borough of Wirral and Mr A M Mascall formerly of Anglian Water (Colchester Division) for their assistance.

8 REFERENCES

1. Muir Wood A M and Fleming C A : Coastal Hydraulics Macmillan, London. 2nd Ed. 1981.

2. Komar P D : Nearshore cell circulation and the formation of giant cusps. Geological Society of America Bulletin. Vol 82, September, 1971.

3. Toyoshimo O : Design of a detached breakwater system. Proc, 14th Conf. on Coastal Engineering Vol II, 1974.

4. Toyoshimo O : Variation of foreshore due to detached breakwaters. Proc. 18th Conf. on Coastal Engineering Vol III, Cape Town, 1982.

5. Cortemeglia G C, Lamberti A, Liberatore G, Stura S and Tomasicchio U : Effects of harbour structures on shoreline variations along the coast of Italy. P.I.A.N.C Bulletin Vol II, No. 39, 1981.

6. Aminti P, Lamberti A and Liberatore G : Experimental studies on submerged barriers as shore protection structures. Proc. Int. Conf. on Coastal and Port Engineering. Colombo, 1983.

7. Fried I : Protection by means of offshore breakwaters. Proc 15th. Conf. on Coastal Engineering, Vol II, Honolulu, 1976.

8. Low Cost Shore Protection : Final Report on the shoreline erosion control demonstration program, U.S Army Corps. of Engineers, 1981.

9. Zwamborn J A, Fromme G A W and Fitzpatrick J B : Underwater mound for protection of Durban's Beaches. Proc 12th Conf. on Coastal Engineering, Vol II, Washington, 1970.

10. Owen M W : Private communications on Rhos-on-Sea breakwaters, 1979.

11. Barber P C and Davies C D : Offshore breakwaters - Leasowe Bay. Proc I.C.E, Part I, 77, February 1985.

12. Pethick J S : Investigation of proposed breakwater efficiency : Dengie Peninsula, Essex. Hull University Report, 1983.

13. Johnson J W, Fuchs R A and Morison J R : The damping action of submerged breakwaters. Trans. Am. Geophys Union. Vol 32, No. 5, October, 1951.

14. Raman H, Jothi-Shakar N and Dattatri J : Submerged breakwaters. J. Central Board of Irrigation and Power, New Dehli, India. Vol 34, No 2, April 1977.

15. Nakamura M, Shiraishi H and Sasaki Y : Wave damping effect of submerged dyke. Proc. 10th Conf. on Coastal Engineering, Vol II, Tokyo, 1966.

16. Milne-Dick T and Brebner A : Solid and permeable submerged breakwaters. Proc 11th Conf. on Coastal Engineering, Vol II, London, 1968.

17. Dean W R : On the reflection of surface waves by a submerged plan barrier. Proc. Cam. Phil. Soc. 41 Part 3, 1945.

18. Dattatri J, Raman H and Jothi-Shankar N : Performance characteristics of submerged breakwater. Proc. 16th Conf. on Coastal Engineering Vol II, Hamburg, 1978.

19. Kabelac O W : Model tests of coastal protective structures in USSR. J. Waterways Div., ASCE 89(1), 1963.

20. Goda Y, Takeda H and Moriya Y : Laboratory investigation of wave transmission over breakwaters. Report of the Port and Harbour Research Institute, No 13, April 1967.

21. Goda Y : Reanalysis of laboratory data on wave transmission over breakwaters. Report of the Port and Harbour Research Institute, Vol 18, No 3, September 1969.

22. Seelig W N : Effect of breakwaters on waes : laboratory tests of wave transmission by overtopping. Proc. Conf on Maintenance and Performance of coastal structures, ASCE, Alexandria, 1979.

23. Allsop N W H : Low crest breakwaters, studies in random waves. Proc. Conf. on Maintenance and Performance of Coastal Structures, ASCE, Arlington, 1983.

24. Sato S and Tanka N : Artificial resort beach protected by offshore breakwaters and groins. Proc. 17th Conf. on Coastal Engineering, Vol II, Sydney, 1980.

25. Rosen D S and Vajda M : Sedimentological influences of detached breakwaters. Proc. 18th

Conf. on Coastal Engineering, Vol III, Cape Town, 1982.

26. Mimura N, Shimizu T and Horikawa K : Laboratory study on the influence of detached breakwater on coastal change. Collected papers of the Dept. of Civil Engineering, University of Tokyo, Vol 21, 1983.

27. Pelnara-Considene R : Essai de Theorie de l'Evolution des Formes de Rivage en Plages de Sable et de Galets. 4th Journees de l'Hydraulique. Les Energies de la Mer, Question III, Rapport No 1, 1956.

28. Ozasa H and Brampton A H : Mathematical modelling of beaches backed by seawalls. Coastal Engineering, 4, 1980.

29. Komar P D and Inman D L : Longshore sand transport on beaches. J. Geophysical Research, 75, No 30, 1970.

30. Kraus N C, Harikai S and Kubota S : A numerical simulation of shoreline evolution at Oarai beach. Proc. 28th. Japanese Conf. on Coastal Engineering. JSCE (1981). (In Japanese).

31. Brampton A H, Franco L and Noli A : Mathematical model of shoreline north of River Arno (Pisa). Estratto da 'Idro tecnioca' n.1. February 1984.

32. Bakker W T : The dynamics of a coast with a groyne system. Proc. 11th Conf. on Coastal Engineering Vol I, London, 1968.

33. Perlin M and Dean R G : A numerical model to simulate sediment transport in the vicinity of coastal structures. U.S Army, Corps of Engineers. Report 83-10. May 1983.

34. Penny W and Price A : The diffraction theory of sea waves by breakwaters. Phil. Trans. Roy. Soc. A 244, 1952.

35. Treloar P and Nagle B : The design of an overtopping breakwater. Proc. 17th Conf. on Coastal Engineering, Vol II, Sydney, 1980.

36. Allsop N W H, Bradbury A P, Poole A B, Dibb T E and Hughes D W : Rock durability in the marine environment. Hydraulics Research Report

No SR 11, 1985.

37. Powell K A and Allsop N W H : Low crest breakwaters, Hydraulic performance and stability. Hydraulics Research Report No SR 57, 1985.

38. Gilbert G and Brampton A H : The solution of two wave diffraction problems using integral equation. Hydraulics Research Report No IT 299, 1985.

39. Smallman J V : A mathematical model of wave diffraction and overtopping by a series of offshore breakwater. Hydraulics Research Report No 59, 1985.

40. U.S Army Corps of Engineers : Shore Protection Manual. 4th Edn. 1984.

41. Copeland G : A practical alternative to the mild slope wave equation. Coastal Engineering 9, 1985.

DDB 11/86

Figures

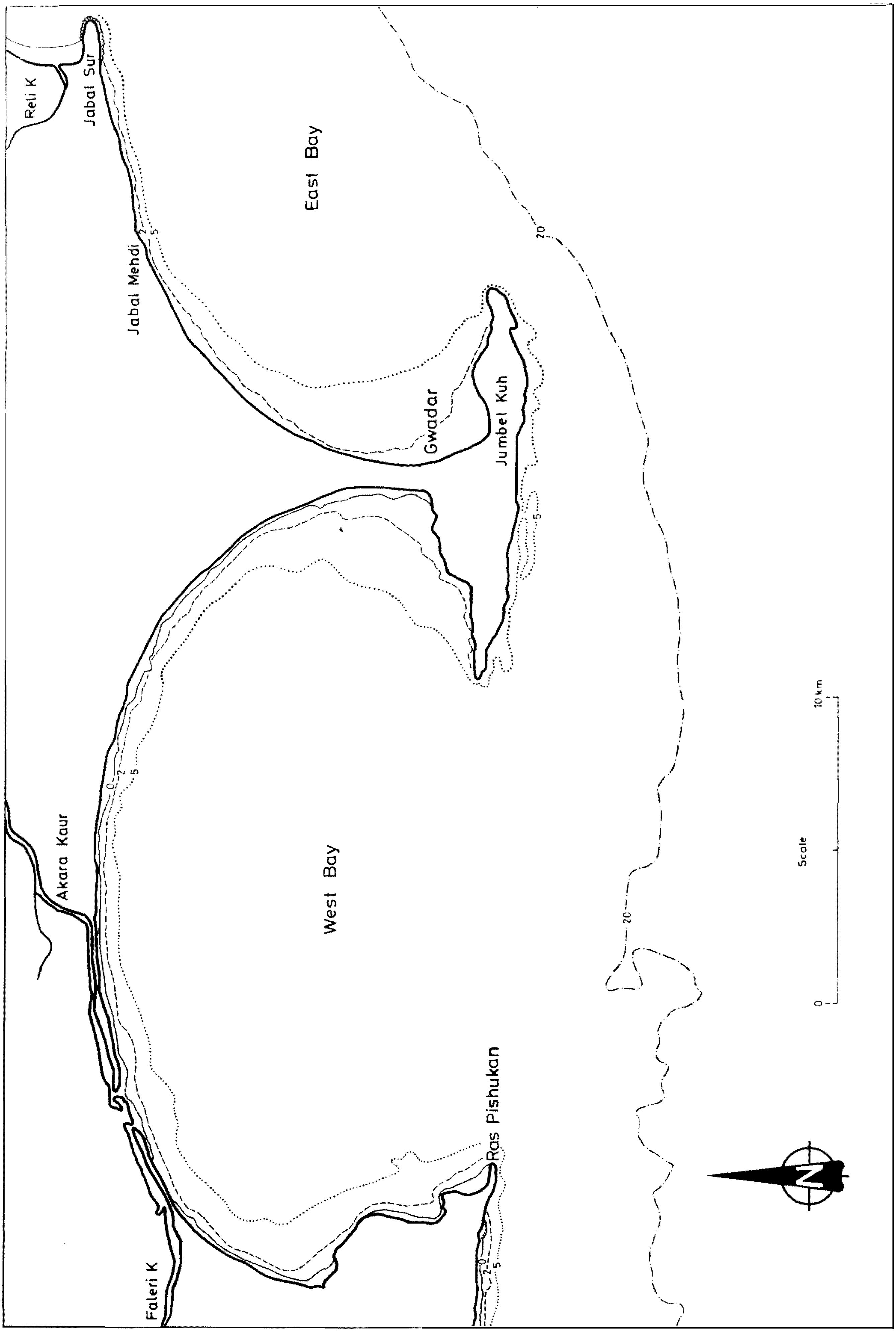

Fig 1 Tombolo at Gwadar, Pakistan

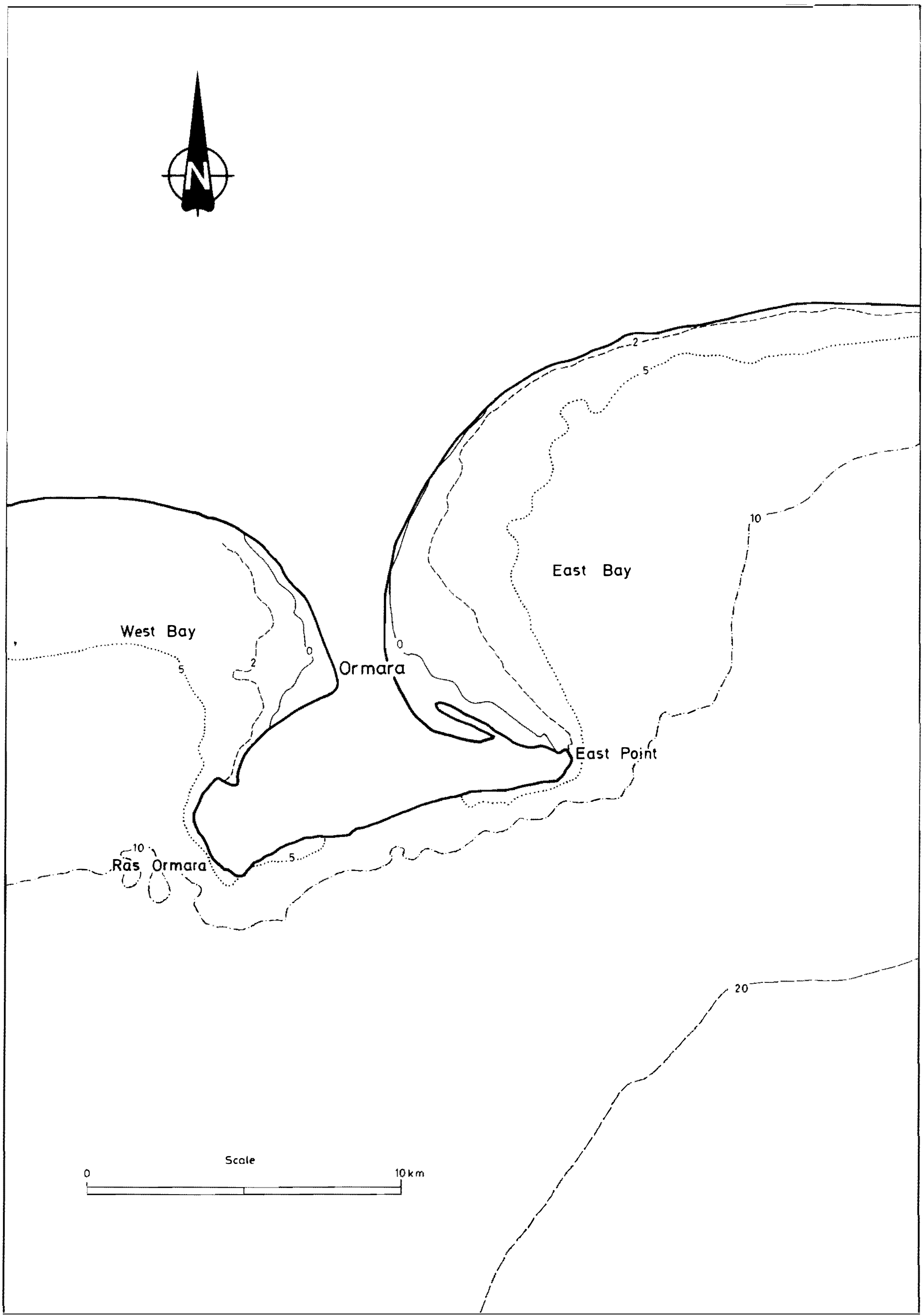

Fig 2   Tombolo at Ormara, Pakistan

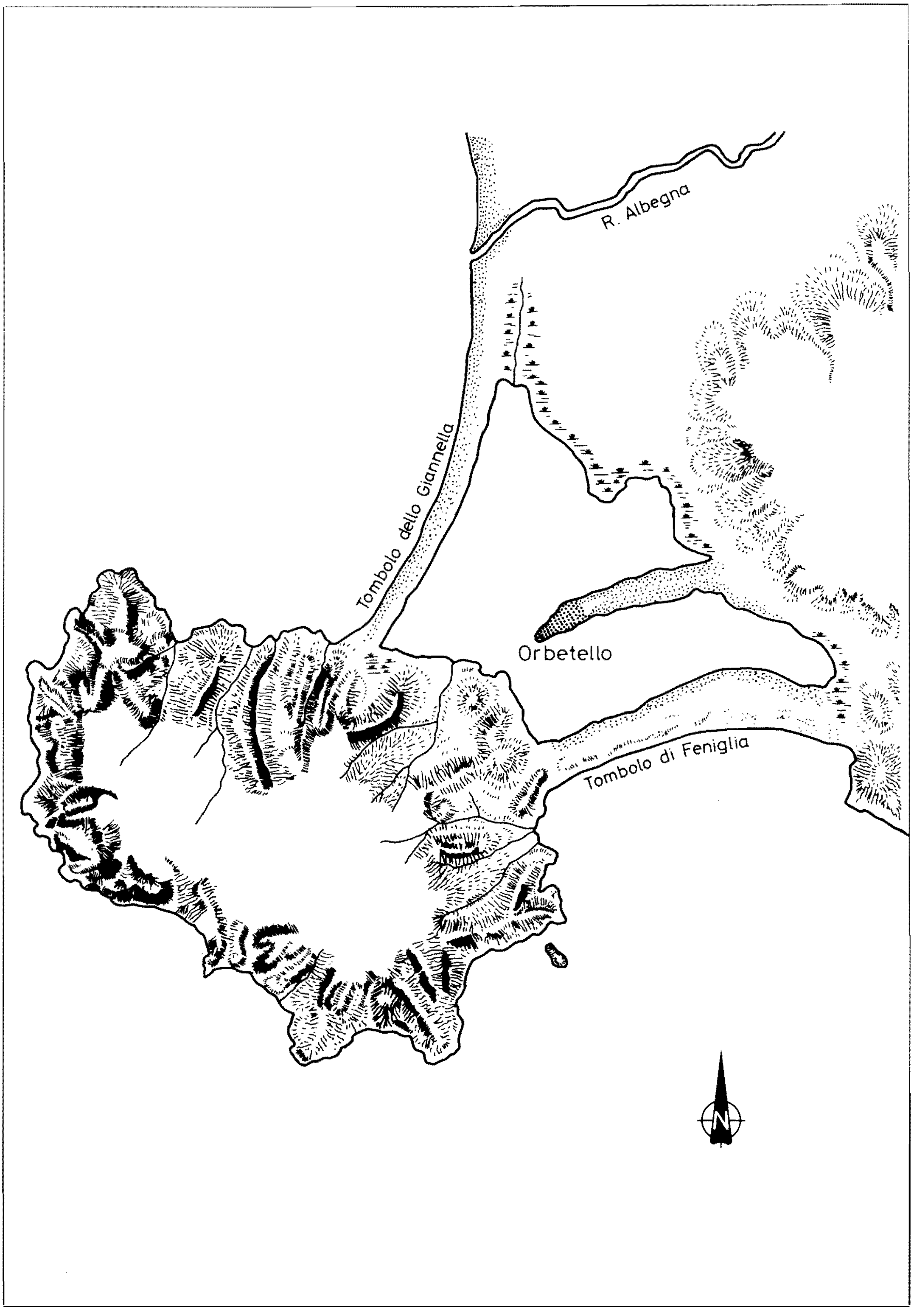

Fig 3 Tombolo at Orbetello, Italy

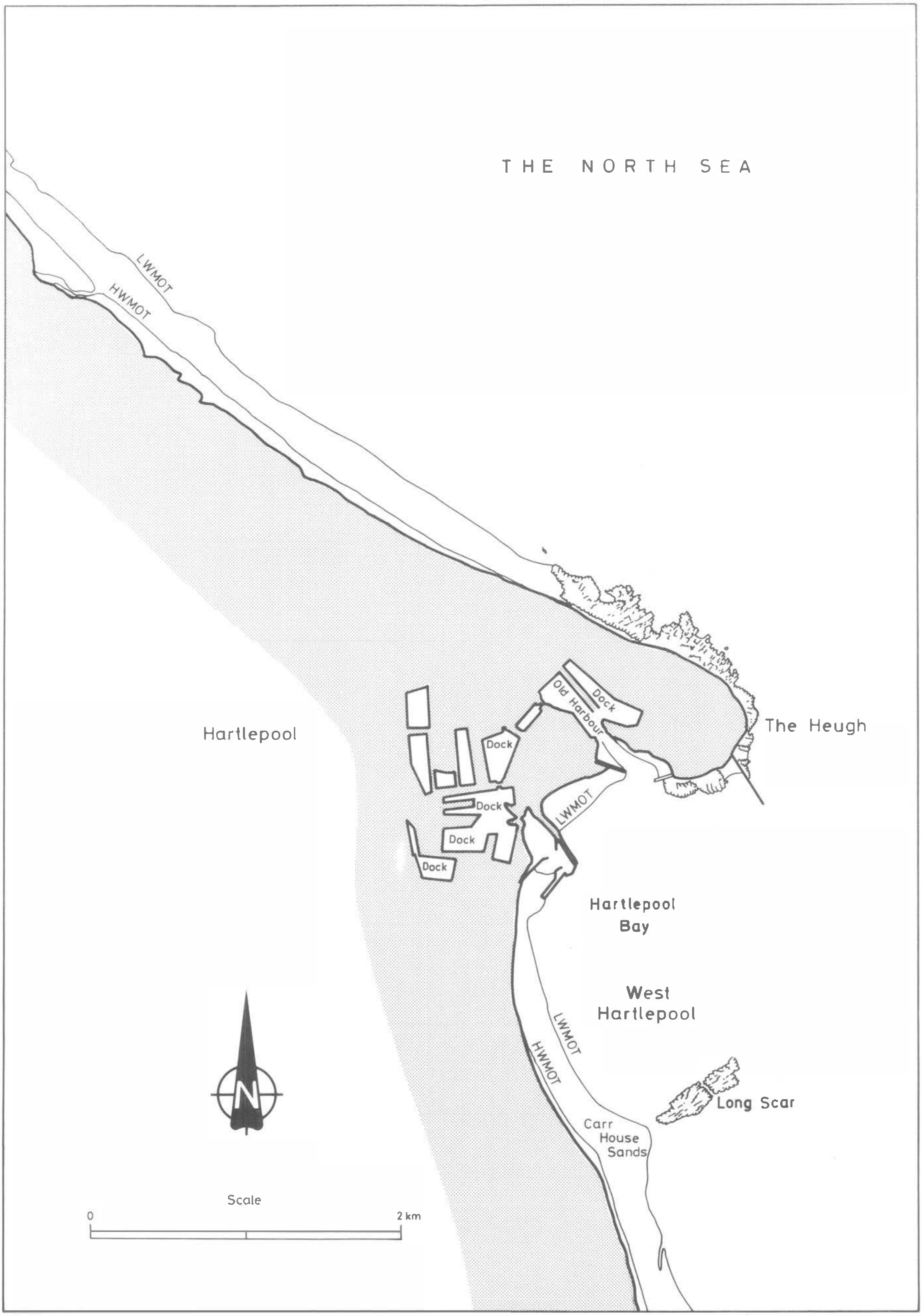

Fig 4 Tombolo at Hartlepool, U.K.

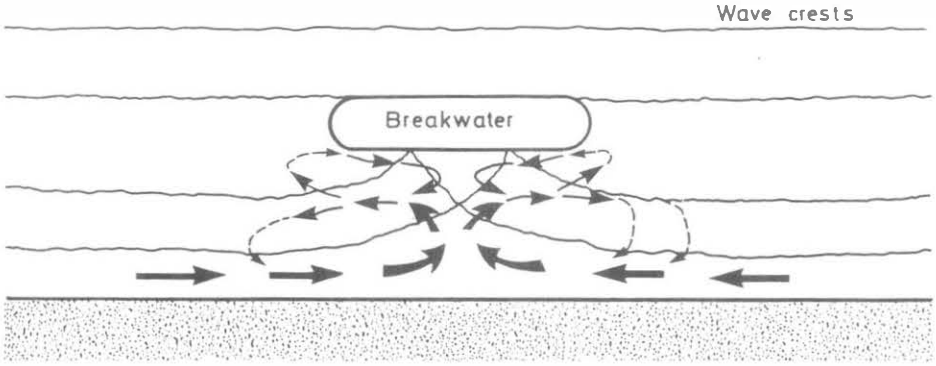

Fig 5a Current system at initially straight coastline

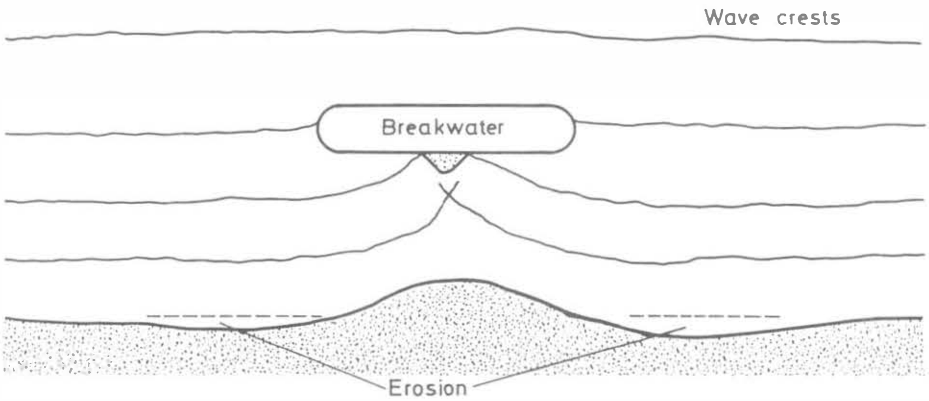

Fig 5b Formation of beach cusp

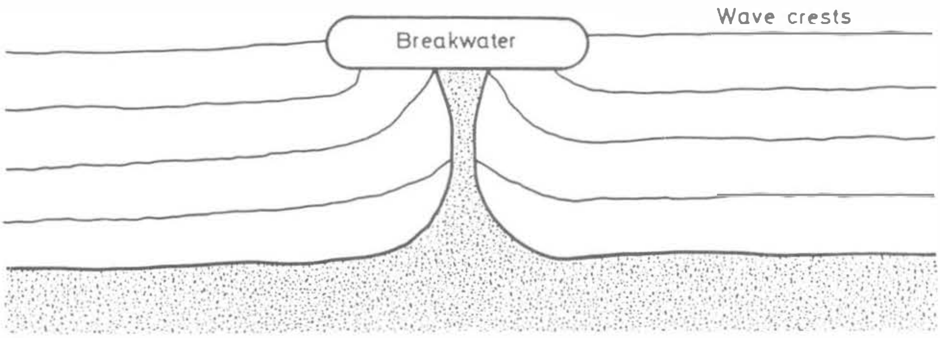

Fig 5c Formation of full tombolo

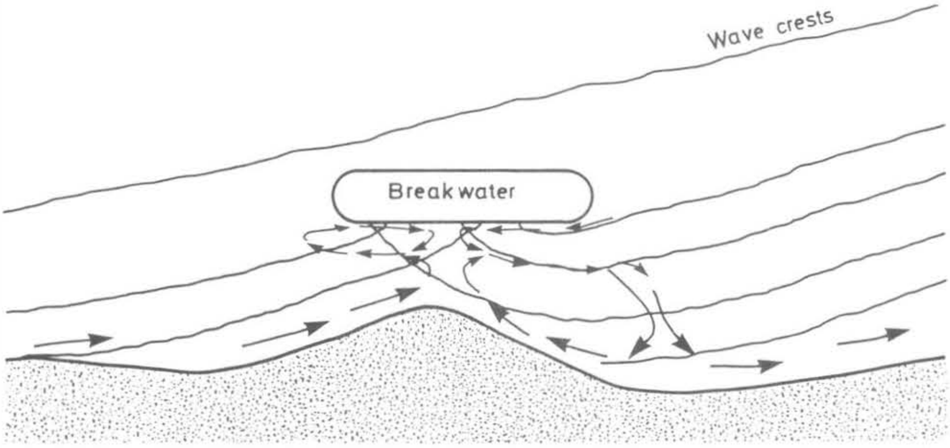

Fig 5d Current pattern where offshore breakwater acts as a groyne

Fig 5 Tombolo formation

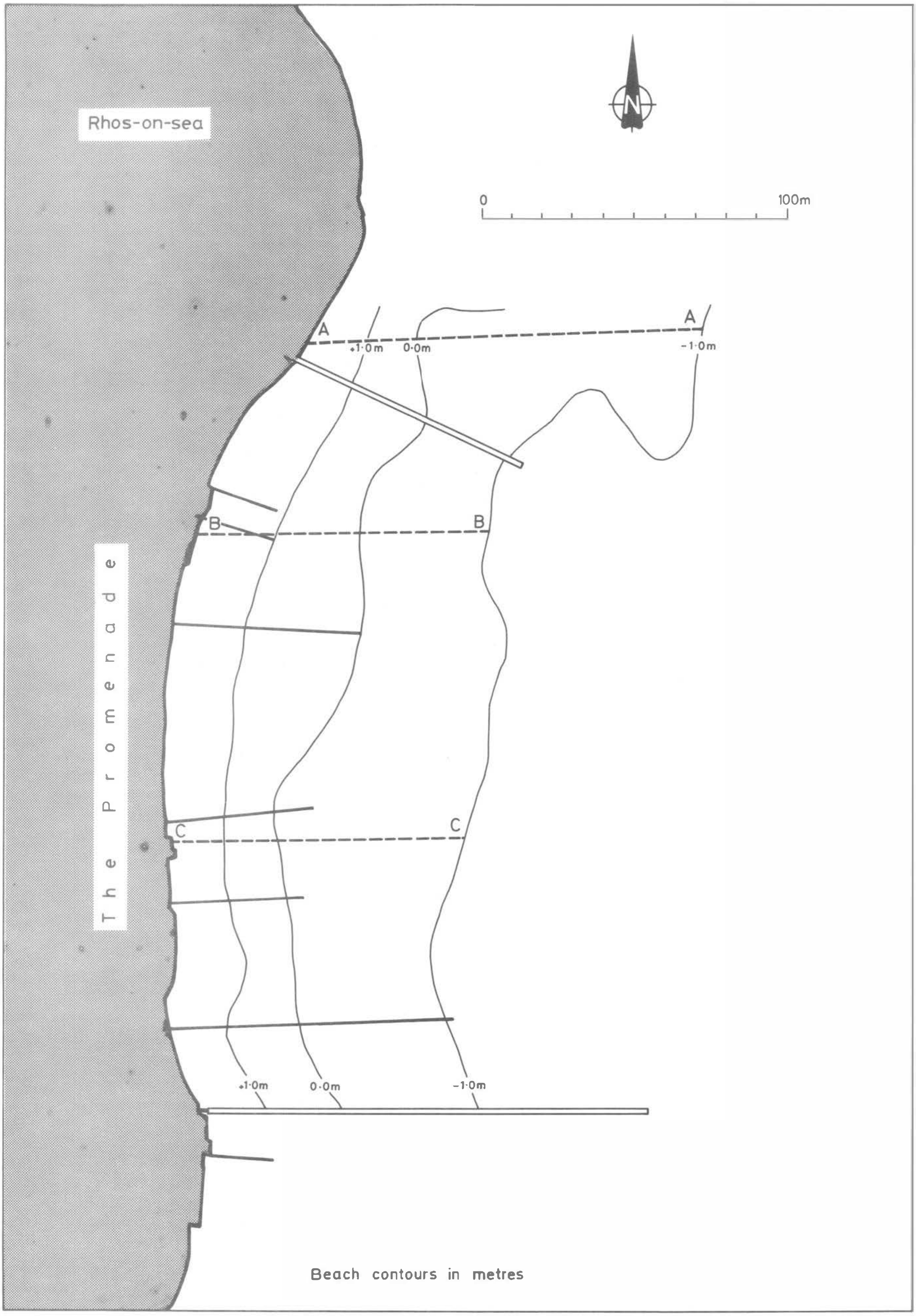

Fig 6a Plan of Rhos-on-Sea before breakwater (February 1980)

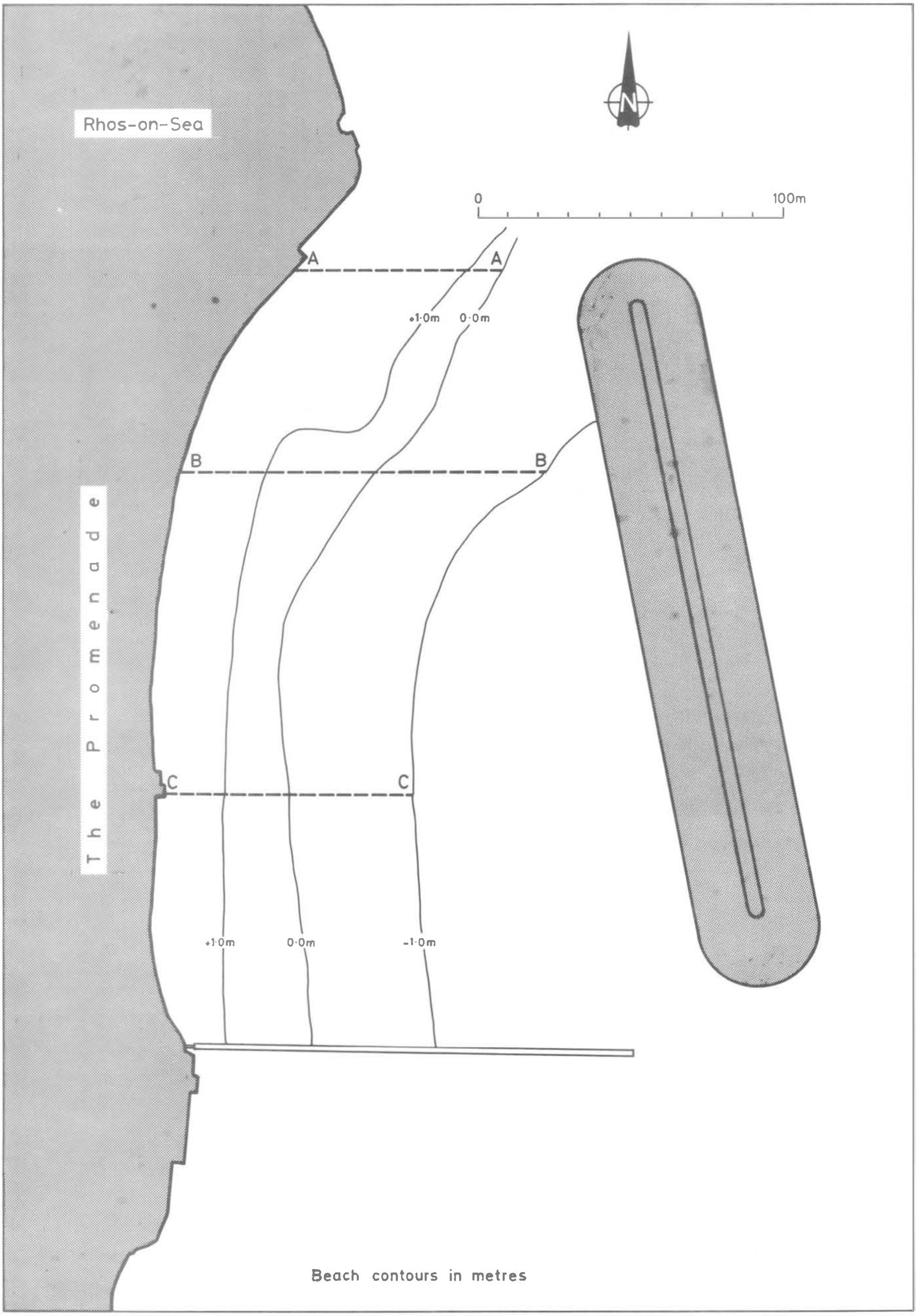

Fig 6b Plan of Rhos-on-Sea after breakwater (July 1984)

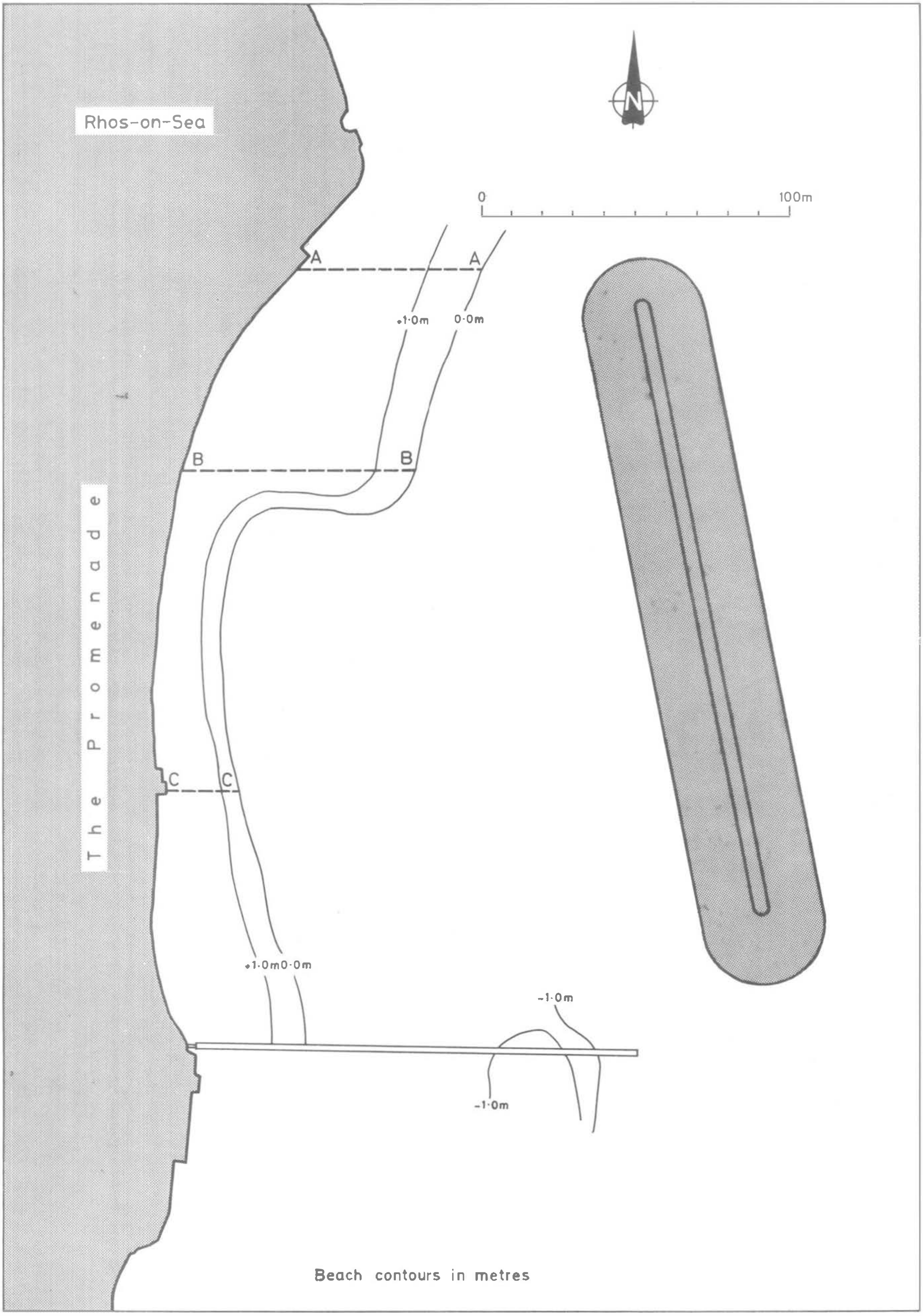

Fig 6c Plan of Rhos-on-Sea after breakwater (September 1985)

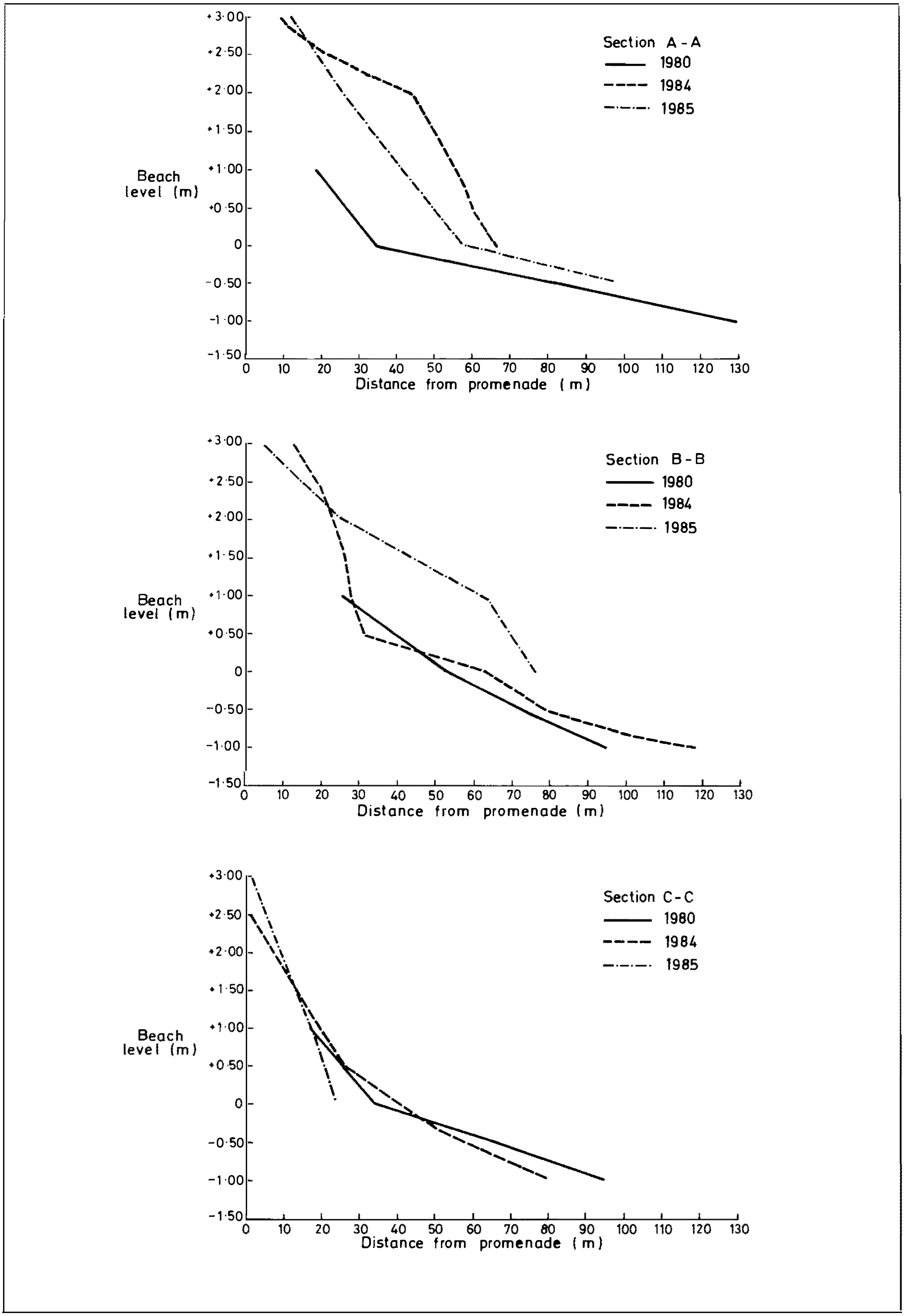

Fig 7 Rhos-on-Sea profiles

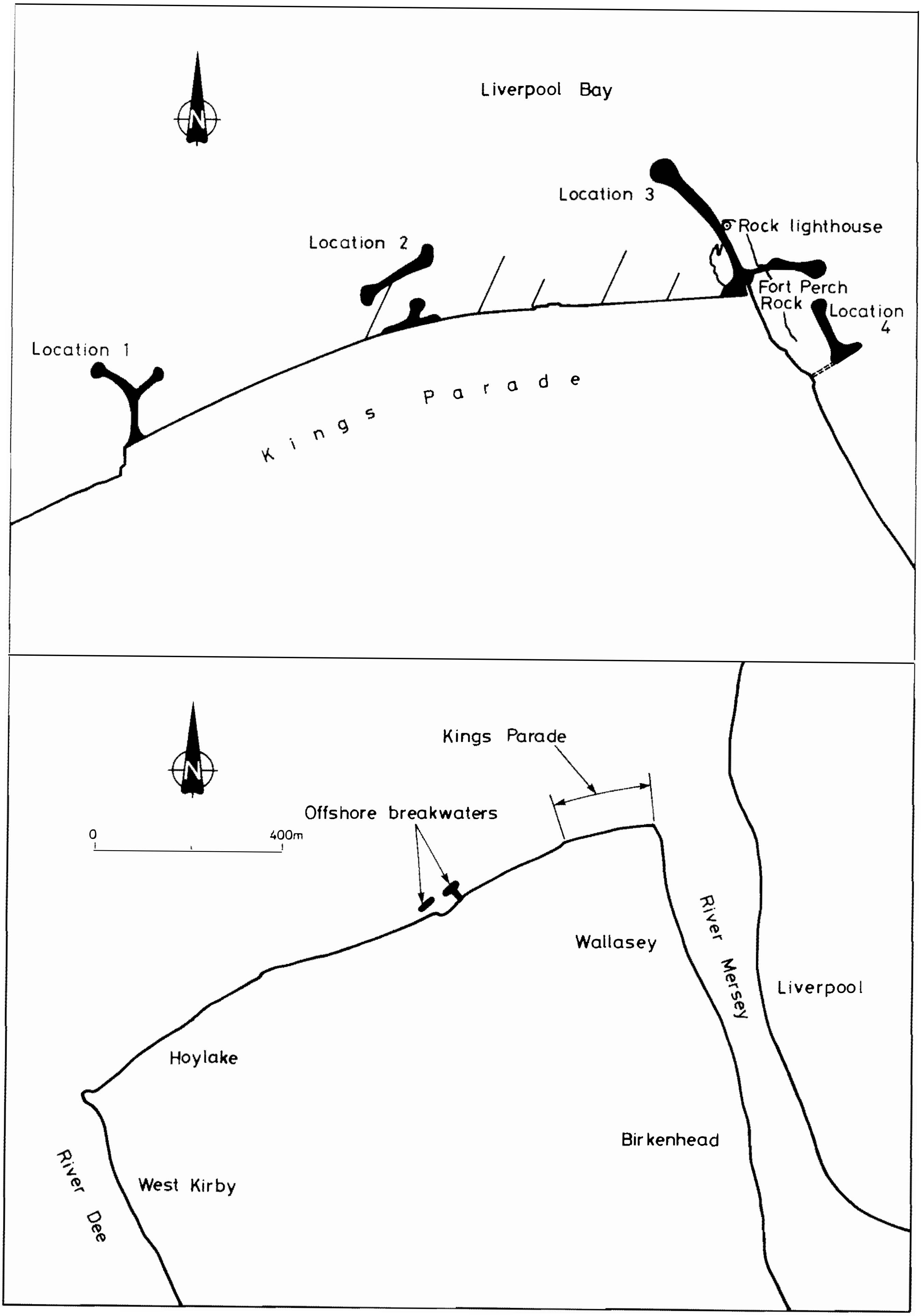

Fig 8 Kings Parade breakwater layout and location

Plates

Plate 1   Offshore reef, Dorset Coast

(Photograph copyright Colour Library International)

Plate 2 Offshore breakwater, Rhos on Sea

Plate 3 Offshore breakwaters, Leasowe Bay

(Photograph copyright Sealand Aerial Photography)