West Bay Coastal Defence and Harbour Improvement Scheme, Dorset

The development of a Beach Management Strategy for East Beach

Report EX 4226
August 2000

HR Wallingford

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Contract

This report describes the work commissioned by Posford Duvivier as a sub contract to the West Bay Coastal Defence and Harbour Improvement Scheme commissioned by West Dorset District Council whose representative was Mr Keith Cole, Engineering Manager. The Posford Duvivier representative was Mr Ian Haken. The HR Wallingford Job Number was CFR 2819. The work was undertaken by Miss Aurora Orsini and Mr Ian Mockett. The report was written by Mr Ian Mockett. The HR Wallingford Project Manager was Eur Ing Paul Sayers.

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Summary

West Bay Coastal Defence and Harbour Improvement Scheme, Dorset

The development of a Beach Management Strategy for East Beach

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West Dorset District Council as part of the on-going West Bay Coastal Defence and Harbour Improvement Scheme in Dorset, UK, commissioned both HR Wallingford and Posford Duvivier to develop a specific Beach Management Plan for the scheme. HR Wallingford was responsible for beach modelling and developing the beach management strategy. Posford Duvivier was responsible for developing the implementation strategy and had overall project management of the study.

This report outlines the modelling undertaken to develop a beach management strategy for East Beach. East Beach is located at the western end of Chesil Beach, which stretches eastward to Portland and is designated as a SSSI (Site of Special Scientific Importance). In recent times the importance of a robust Beach Management Plan has become increasingly apparent as on a number of occasions flooding has resulted from flow through or over the shingle bank. With the construction of the preferred scheme, the issue of shingle intrusion into the harbour entrance also requires attention and the Beach Management Plan needs to consider mitigation measures to reduce the risk of excessive dredging of by-passed material.

Results from three types of model were used during the study, including a physical model and two numerical models. One numerical model considered the short term storm response of the beach and the other considered the long term evolution of the beach plan shape.

The study assumed a design profile for the shingle bank comprising a 10m wide crest at +7.5mODN with a 1in 6 leeward slope. Seaward of the crest, the beach sloped at 1 in 3 that then merged into a 1 in 8 slope. The offshore bathymetry sloped at 1 in 100. The design profile is based upon beach cross sections provided by WDDC in April 1999 in preparation for the physical model.

The conclusions of the study are:

- Modelling of the long-shore and cross-shore processes have indicated that the defined beach width (distance from the base line to the mean water level) should be maintained within the following limits:

  - Extreme Lower Limit = 120m
  - Lower Limit = 130m
  - Upper Limit = 150m
  - Extreme Upper Limit = 160m

Summary continued

- If the do nothing scenario is adopted then there is a high chance that an unacceptable volume of shingle will enter the harbour entrance. This will require excessive dredging to clear the entrance, with attendant environmental and economic consequence.

- The longshore drift can vary in both directions from month to month. It is therefore difficult to identify a set amount to re-cycle or renourish per annum. An observational approach is proposed to reduce the likelihood of beach material by passing East Pier and entering into the harbour. Even with this approach some dredging will still inevitably be required.

- Narrowing the beach by extraction of material in the vicinity of the harbour entrance, in order to minimise the risk of sediment ingress into the harbour, will necessarily increase the risk of local breaching and overtopping of the beach. This is best managed by careful re-use of the re-cycled material. It is suggested that the crest of the berm is locally widened/raised where there is significant room to implement this, rather than re-cycling to a point some distance to the east of the study frontage.

- The results discussed are based on previously occurring wave conditions, and there is no guarantee that similar wave conditions will continue in the future. It is therefore important that the beach is carefully monitored so that robust management decisions are made based on the real time evolution of the beach. As further information is collected, a better understanding of the beach will allow further enhancements to the modelling and decision making process.

- In terms of climate change, the main impact will be any change in storminess (i.e. magnitude of storms and direction) rather than sea level rise as long as the beach is maintained at a width higher than the minimum beach width.

- In terms of large swell events, which are regularly observed in Lyme Bay, the beach will perform satisfactorily as long as it is maintained at a width higher than the minimum beach width. However, after a significant swell event some re-nourishment or re-profiling work is likely to be required.

The recommendations of the study are:

1. The beach width should be maintained between 130m and 150m, but should not exceed 160m or be less than 120m. The defined beach width is measured from the model base line.

2. An observational approach to beach management should be adopted with re-cycling and re-nourishment being undertaken when required. Care needs to be taken that to ensure the risk of flooding is not increased to an unacceptable level when re-cycling material.

3. The economic and environmental consequences of re-cycling or allowing material to by-pass East Pier should be carefully assessed in order to determine what the best beach management policy is.

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1. INTRODUCTION

West Dorset District Council as part of the on-going West Bay Coastal Defence and Harbour Improvement Scheme in Dorset, UK, commissioned both HR Wallingford and Posford Duvivier to develop a specific Beach Management Plan for the scheme. HR Wallingford was responsible for beach modelling and developing the beach management strategy. Posford Duvivier was responsible for developing the implementation strategy and overall project management of the study.

This report outlines the modelling undertaken in order to develop a beach management strategy for East Beach only. East Beach is located at the western end of Chesil Beach, which stretches eastward to Portland and is designated as a SSSI (Site of Special Scientific Interest). In recent times the importance of a robust Beach Management Plan has become increasingly apparent as on a number of occasions flooding has resulted from flow through/over the shingle bank. With the construction of the preferred scheme, the issue of shingle intrusion into the harbour entrance also requires attention and the Beach Management Plan needs to consider mitigation measures to reduce the risk of excessive dredging of by-passed material.

For the purposes of this study, East Beach is defined as the beach from East Pier to the end of the shingle bank where it joins the cliffs. All beach width measurements are taken from the baseline used in the BEACHPLAN model. All beach width measurements in this report refer to a cross section approximately 50m East of East Pier. A reference grid and plan is shown in Figure 1.

This report uses the most up to date techniques to evaluate the likely performance of the beach system and to predict how it will perform in the future. However, the performance will be closely limited to future wave conditions, thus for beach management to be carried out in the most effective manner continuous review will be required aided by on-going monitoring and management.

2. TERMS OF REFERENCE

The Terms of Reference for the study are to:

1. Develop a beach management strategy for East Beach in order to fulfil flood protection and harbour operation needs.

2. Consider the impact of extreme events such as swell waves and climate change on beach performance.

3. DESCRIPTION OF MODELLING UNDERTAKEN

Results from three types of model were used during the study, including a physical model and two numerical models: one considering short term storm response and other considering the long term evolution of the beach plan shape. The aspects of these three models are summarised below. Further details of the methods are given in the relevant HR Wallingford Reports referenced where appropriate.

3.1 Physical model

The main issues considered in the physical model of relevance to this study were:

- The storm response of East Beach for a range of sea conditions.
- Mitigation measures to reduce material by-passing the East Pier Extension.

The physical modelling of East Beach is described fully in HR Report EX4064 (HR Wallingford 2000a).

3.2 Storm response prediction

The storm response model used to assist in the derivation of the coastal defence and harbour improvement strategy plan for West Bay (HR Wallingford 2000b) was adjusted using additional knowledge gained from the physical model study (HR Wallingford 2000a). The technique is described in the Failure of East Beach letter report (HR Wallingford 1998a) presented to WDDC during the strategy study. The following items were assumed in the modelling:

- **Assumed beach geometry and sediment grading**
  A typical design profile of East Beach was taken from cross sections provided by WDDC in April 1999 as part of the preparation for the physical model. The typical profile had a 10m crest at +7.5mODN with a 1in 6 leeward slope. Seaward of the berm, the beach sloped at 1 in 3 merging into a 1 in 8 slope. The offshore bathymetry sloped at 1 in 100.

  The typical sediment grading was defined by a site investigation on East Beach carried out in November 1996 for WDDC by Exploration Associates. The typical D_{n50} of the samples was approximately 8mm.

- **Failure Criteria.**
  Described below in Section. 4.1.

- **Calibration of the SHINGLE model using the physical model observations.**
  Two wave and water level conditions were tested in the physical model with a joint return period of 100 years and 2000 years with a wave direction of 220 degrees north. These profiles were compared with the predicted lines for the same wave condition from the uncalibrated HR Wallingford SHINGLE model.

  The profiles measured in the physical model showed a significant larger rate of cutback than the SHINGLE model. This was mainly because the time scaling of the longshore component in the physical model was 8 times faster than the crossshore time scaling. Therefore, the measured profile was adjusted such that the loss of area per hour matched the true longshore drift. After adjustments, the comparison showed that the uncalibrated SHINGLE model under-predicted the shoreward excursion of the 2% wave run up position by approximately 3 to 5m (Figure 2 and 3). The conservative decision was made to correct the predicted position of the 2% wave run-up limit given by the SHINGLE runs used in the development of the storm response model by 5m landwards. It is worth noting that in this report, wave run up is referred to in terms of its shoreward excursion rather than elevation.

- **Further correction of the SHINGLE model to account for bi-modal wave conditions observed in Lyme Bay.**
  Flume studies on the response of shingle beaches to bi-modal conditions (i.e. the combination of swell and wind sea conditions) showed that as the swell condition starts to dominate over the wind-sea condition (in terms of their relative energy inputs to the storm) wave run-up increases (Hawkes et al, 1997). For a storm consisting only of swell energy, the average increase in the wave run-up distance when compared to a storm of equal energy but consisting entirely of wind-sea has been estimated as 40 percent (Figure 4).

  In Lyme Bay, bi-model conditions are common with exposure, well waves remotely generated in the Atlantic Ocean. It is, therefore, important to consider bi-model conditions when predicting wave run-up and overtopping at West Bay. The SHINGLE model is validated only for wind-sea wave conditions. So to represent the truer wave condition at West Bay, the results from the SHINGLE model have been corrected to allow for the influence of bi-model conditions.

The flume studies considering bi-model conditions were undertaken in a controlled manner. However, in the field it is difficult to accurately assess bi-modal conditions plus their true impact on the structural response of the beach. Therefore, a linear relationship between the percentage increase in wave run-up and bi-modal conditions was derived, with 40 percent being the worst case scenario for 100% swell. For example, if the storm contains 50% swell energy and 50% wind-sea energy the wave run-up distance has been enhanced by 20%.

A relationship between the predicted wave height assuming wind-sea condition only and the proportion of expected swell for such a condition was then derived using the predictions in the Swell and Bi-Modal Atlas for England and Wales (HR Wallingford 1997a). This relationship is shown in Figure 5.

To take account of the impact of bi-modal incidents sea conditions on wave run-up the ε wave run-up distance predicted by SHINGLE was enhanced by a factor, F, given as:

$$F = -0.0783.H_{s(wind)} + 1.4867$$

Where $H_{s(wind)}$ is the wind sea significant wave height.

For example, if the wave run up generated by a 2m (Hs) wind-sea is 5m past the seaward crest of the beach, then the wave run-up enhancement factor to take account of the likely swell component of that particular seastate is 1.33 (i.e. -0.0783*2+1.4867). The maximum wave run-up distance, therefore, for the bi-modal condition likely to occur at West Bay is 6.7m (i.e. 1.33*5) plus 5m to account for the calibration of the SHINGLE model against the physical model results (i.e. 6.65+5.0=11.7m).

3.3 Beach planshape prediction

As part of the main Strategy Study a beach planshape model, BEACHPLAN, was set up and calibrated against historical data. This model was then further adjusted to take account of the East Pier Extension using data provided by the physical model. The calibrated adjusted model was then used to establish the performance of simple beach management strategies.

The BEACHPLAN modelling undertaken for West Bay is described fully in HR Report EX4137 (HR Wallingford 2000c).

4. ESTABLISHING A MINIMUM BEACH WIDTH

This section considers the minimum beach width along East Beach that will provide an acceptable standard of flood protection.

4.1 Definition of failure criteria

When waves break, material is carried up the beach as bed and suspended load. Part of the uprush percolates into the beach, causing the volume and velocity of the backwash water to be reduced. The differential between the uprush and backwash energy causes sediment to be either deposited or removed near to the limit of the wave run-up. The nett result is an accumulation or loss of material on the beach. In storm conditions, these processes will continue until equilibrium between the uprush and backwash energies is reached, resulting in the formation of a new upper beach profile.

The performance of the beach as a flood defence is then determined by its ability to maintain the wave run-up limit seaward of the crest. If wave run-up passes landward of its crest, then overtopping will be observed and any material still in motion will be washed over the crest of the beach.

Shingle beaches often dissipate energy as water passes over their crest, restricting shingle deposition to the seaward side of the beach. Under these circumstances any shingle that is pushed or carried to the crest by the uprush water is deposited at the crest, thus causing the crest level to increase. Combined with the high permeability of the beach, this crest building process usually ensures that most of the incident wave energy is dissipated before it reaches the lee face of the beach crest. However, when a critical combination of freeboard, crest width and wave condition is reached, the beach will become susceptible to severe overtopping resulting in either crest rollback, crest lowering or formation of shingle *'fans'* behind the crest.

Historically, the leeward movement of the beach crest at West Bay has been limited by the fixed position of a number of buildings and the tarmac yard located 10 to 20 metres behind the crest (at an approximate level of 4.5 to 5.5 mODN). These features hinder the movement of the shingle beach, limiting the ability of the crest to roll back.

From the above it can be seen that the likelihood of failure occurring depends on two aspects:

- The prevailing beach width prior to storm attack
- The horizontal excursion of the wave run-up experienced during that attack.

The impact of beach width on the likelihood of failure is relatively simple; a narrow beach offers less protection than a wider one. The reducing residual strength of the beach as its width reduces has been incorporated into the analysis as described below.

When considering the likelihood of "failure" as a function of wave run-up, four possible scenarios have been considered. The relationship between each of these scenarios and the probability of failure associated with that scenario has been quantified and used to predict the likelihood of a breach occurring during any particular storm condition and beach width position (see Figure 6).

<u>Scenario 1</u>
2% wave run-up confined to the region seaward of the crest edge. It is reasonable to assume that this natural movement of the beach will have no impact on the crest.

Assigned probability of failure = 0.0

<u>Scenario 2</u>
2% wave run-up reaches or exceeds the seaward edge of the crest, but remains within the crest region. It is unlikely that failure will occur because crest building will occur some distance in front of the 2% wave run up point. The structural stability of the beach will be maintained.

Assigned probability of failure = 0.0

<u>Scenario 3</u>
2% wave run-up passes the leeward edge of the beach crest. Failure will start to become more certain as the amount of material available to continue the natural roll back of the crest will start to become limited. However, in most cases the crest should be able to still build and maintain its structural stability.

Assigned probability of failure = 0.3

<u>Scenario 4</u>
2% run-up passes onto the hinterland, the depth of material to continue the roll back of the crest will reduce and material will start to fan out over the hinterland. At this point failure of East Beach is certain.

Assigned probability of failure = 1.0

4.2 Prediction of storm response

The SHINGLE model plus the 5m correction was used to provide a base set of run-up values for a wide range of sea conditions using the design profile. These values were then adjusted for the potential swell component that may be expected for that particular wind sea condition. These results were passed through a statistical analysis to form a quadratic surface (Figure 7, results are for a beach width of 130m) and generate a mathematical description of wave run-up as a function of wave height, water level and initial beach width. This equation takes the following form:

$$\text{Wave run-up} = C_1 + C_2.\ H_s + C_3.HWL + C_4.\ H_s^{\ 2} + C_5.HWL.H_s + C_6.HWL^2$$

where Wave run-up = the horizontal position of the run-up relative to the seaward crest edge
HWL = Still water level (mODN)
Hs = Near shore wave height (m)
$C_1 ... C_6$ = Quadratic constants

This approach was used to develop a similar formula for a range of initial beach widths from 110m to 150m. As the beach width reduces, the crest will typically remain in the same position (unless storm wave action moves it back) causing the foreshore slope to steepen. The SHINGLE model was, therefore, used to predict the sensitivity of the wave run-up limit as beach width reduces and the beach slope steepens. For any particular wave condition, a 10m shift in the high water mark showed a corresponding 7 to 10m shift in the chainage of the wave run-up limit. From this analysis, the conservative assumption was made that for a 10m shift in the beach width, a similar 10m shift in the wave run up position also occurred.

Comparative checks were made on the prediction of the equation and the physical model data, and the results were found to be within the limits expected.

4.3 Consequence of a failure of East Beach

If East Beach fails, flooding in West Bay may be severe.

If a shingle bank is maintained at +7.5mODN throughout a storm is significant overtopping is likely to only occur during a 100 year event. However if a breach occurs and the bank crest lowers to +5.0mODN, then significant overtopping would occur during the 10 times a year event (Table 2). Compared to the cross shore modelling, it is expected that there would be a lowering of the bank and some overtopping during a 13 year event. This value seems reasonable as the overtopping modelling is undertaken assuming a static profile, so as the crest lowers naturally in prototype, the frequency of overtopping tends towards overtopping predicted with the +5.0mODN crest.

For full details of the overtopping calculations please refer to the letter report on Overtopping sent to WDDC (HR Wallingford 1998b)

4.4 Definition of minimum beach width

The traditional method of determining the annual frequency of failure would be to search for the combination of water level and wave height that provides the *'worst case wave run-up'* for a given return period (based on joint return period events). For risk assessment purposes, it is better to work with a set of synthetic data, whose frequency distribution is equivalent to the extreme value distribution derived from the original wave and water level data, such that the worst case wave conditions for that particular structural response are searched for.

Using the 10,000 years of synthetic data derived by the earlier joint probability study (HR Wallingford 1997b), the defined failure criteria, and the equation to describe wave run-up, it has been possible to determine the number of likely failure events in a year for a number of beach widths.

The method used to determine the annual probability of East Beach failing involves three distinct steps:

- **Step 1: Identify the probability of the wave run-up limit stopping at a certain point.**
The wave run-up position for each sea condition listed in the 10,000 years of synthetic data (representing 10000 * 707 high tide events) was determined by using the function derived in Section 4.2. A simple frequency analysis was performed to identify the probability of wave runup limit occurring within a given interval. The probability of occurrence was then defined as follows:

  probability of the wave run-up lying within the given interval per high tide = *no. of events falling within a given interval / the total no. of high tide events*

- **Step 2: Identify the probability of failure assuming a given beach width**
If the wave run-up lies within a given spatial interval, the probability of failure occurring for that particular interval is calculated as follows:

  probability of the wave run-up lying within the given interval per high tide probability of failure occurring given the wave run-up limit is within that interval.*

  Assuming the beach width is fixed, the probability of failure for a particular beach width is given by:

  ∑ probability of the wave run-up lying within the given interval per high tide probability of failure occurring given the wave run-up limit is within that interval.*

- **Step 4: Identify the number of likely breach events in a given year**
The number of likely breaches in a year is then simply given by:

  ∑ probability of failure occurring assuming a given beach width per high tide total number of high tide events in a year (i.e. 707 possible events)*

The calculations for the present day conditions are presented in Table 1. From inspection of the results, a sensible minimum beach width is130m, as maintained, a wider beach offers little further reduction in risk. It is, however, considered that a narrower beach width of 120m may be tolerated for shorter times (Figure 8).

4.5 Performance of the minimum beach width under extreme swell conditions

Until recently swell waves have not really been included in the design of coastal structures. Recent research (Hawkes et al 1997) has, however, shown that swell waves can have a significant impact on coastal structures, especially beaches. Therefore it is important that the impact of these events is considered.

Two swell conditions were selected, the 1 year and 100 year which have a respective significant wave heights of 2.3m and 4.1m and an average wave period of 16s (HR Wallingford 1997a). It should be noted that for swell conditions it is particularly difficult to identify a corresponding water level due to these conditions arising remotely from the site. Therefore, it was assumed that the MHWS water level was appropriate based on the observation that both conditions are likely to happen when there is either a high astronomical tide with a low surge or a low astronomical tide with a high surge.

The SHINGLE model predicted both that the 1 year and 100 year swell event would cause significant damage to East Beach, even with a beach width of 130m the cutback caused significant profile changes extending up to 10m and 30m behind the seaward crest (Figure 9 & 10). (It should be noted that once the landward limit of profile disturbance, as predicted by SHINGLE, migrates beyond the landward edge of the beach crest, the beach can be effectively considered to have failed and subsequent profile predictions are no longer valid.)

An empirical formula developed by Bradbury links wave steepness with a barrier inertia parameter to make a judgement on the likelihood of a shingle barrier breaching (Bradbury, 2000). This formula indicates that East Beach is likely to fail in both swell conditions are considered. However, the formula has only been validated for wave steepness as between 0.032 and 0.057 well above those of typical swell conditions. The wave steepness for the 1 year and 100 year swell event is 0.004 and 0.006 respectively.

Both Bradbury and SHINGLE, therefore, suggest that there is a risk of failure due to swell events when the beach width is 130m. However, its risk is considered acceptable, a conclusion confirmed by the results of the physical model that show little evidence of a breach forming.

From the analysis, it is reasonable to assume that during a swell event there will be significant movement in East Beach. Both numerical techniques are being used outside their current limits and therefore, the best information available is the physical model results, which suggest that the beach will remain intact as long as the 130m beach width is maintained. However after such an event, the beach may require re-profiling or renourishment to reinstate the minimum width of 130m.

4.6 Sensitivity of the cross shore response of the beach to climate change

Sea level rise will have an impact on the hydraulic conditions that reach the East Beach in the form of larger wave heights reaching the beach causing waves to run further up the beach. The impact of sea level rise on flooding, say over 10 years and assuming sea level rising at a rate of 5mm/yr, will be an increase in risk levels of approximately 5 percent (Tables 3 to 7). There will, therefore, be a need to monitor sea level rise and storminess and modified the minimum beach width as necessary. It is recommended that this review is done on a 5 yearly basis.

5. MAXIMUM BEACH WIDTH

This section considers the maximum beach width at which bypassing of material from East Beach into the Harbour entrance is becomes unacceptable.

5.1 Physical model observations

The physical model described in HR Wallingford Report EX4044 (HR Wallingford 2000a) was run for a number of wave and water level conditions from 160^0N. The beach was allowed to build against the East Pier and in all conditions, bypassing of East Pier started when the beach width reached approximately 160m. The results from the physical model were then used to adjust the BEACHPLAN model so that the performance of the East Pier Extension could be better represented in the numerical model.

5.2 Numerical model results

BEACHPLAN was run for a period of 20 years to estimate when shingle material would bypass the East Pier Extension and enter the Harbour entrance. By inspection, it was found that once the beach width reached approximately 150m then some small bypassing was observed, but it was not until the beach width passed 160m that significant bypassing was observed (Figure 11); a result consistent with the observations made in the physical model.

The majority of bypassing usually occurs in years when high nett east to west drifts occur (Figure 12). However, even during periods when the nett longshore drift is eastward away from the pier, some bypassing still occurs as the drift because direction and magnitude fluctuates. For example, as seen in Year 7 plotted on Figure 12, the nett longshore drift is negative but some bypassing still occurred during the year.

5.3 Sensitivity of the long shore response to the beach to climate change

Sea level rise will have minimal impact on the movement of beach material along Chesil Beach. It is more likely that the long term evolution of East Beach will be affected by changes in the direction, frequency and intensity of coastal storms. It is very difficult to estimate, at present, possible changes in storm occurrence, although there are indications that both storm intensity and frequency might increase slightly (Institute of Hydrology, 1994). This topic is presently being investigated but no firm recommendations have yet been made.

A climate change scenario in which westerly winds became more frequent, and south easterly storms less common, would result in a long term erosion problem along East Beach. Alternatively if the trend was reversed then shingle intrusion into the harbour would increase dramatically. Current thinking suggests that there may be a 20% chance of a prolonged south-easterly drift, perhaps a 60% chance of no change to the existing drift pattern and a 20% chance of a north-westward drift setting in. Drift patterns will need to be monitored carefully to help with the operational management of the beach but have little impact on selecting the maximum and minimum beach widths.

6. BEACH MANAGEMENT STRATEGY

A beach management strategy for East Beach needs to optimise recycling and renourishment activities to reduce the risk of flooding and the risk of significant bypassing of East Pier. Modelling of the beach indicates a number of possible beach plan shapes and their potential implications for coastal management, which are shown in Figure 13.

The strategy, therefore, needs to consider maintaining the beach within set limits. From the cross shore and long shore modelling the following limits have been derived:

- Extreme Lower Limit = 120m
- Lower Limit = 130m
- Upper Limit = 150m
- Extreme Upper Limit = 160m

It is difficult to manage a beach, which is very mobile and can fluctuate in beach width by 20m between a matter of months without active management, with periods when the beach will be under nourished (i.e. less than 120m wide), and times when there may be significant bypassing into the harbour.

HR Wallingford Report EX 4137 considered some simple strategies for recycling at a constant rate of 5,000m^3/yr and 10,000m^3/yr in order to maintain the beach width within the acceptable limits (HR Wallingford, 2000c). Both of these strategies reduced the total amount of shingle entering the harbour from the 'do nothing' scenario of 23,000m^3 to 19,000m^3 and 16,000m^3 respectively over a period of 20 years (Figure 14). The greatest bypassing requirements occur when the nett longshore drift is between 20,000m^3/yr and 40,000m^3/yr with the majority of east to west drift taking place in a shorter period of 1 to 2 months. Even with re-cycling rates of 10,000m^3/yr, beach movements can not be controlled. During these periods of high and short duration longshore drift rates, a more intensive recycling regime is required.

Therefore, to minimise shingle intrusion and flood risk, an observational approach is recommended to assist in managing the beach width. This approach consists of a simple three point action plan:

Measured Beach Width	**Action**
Beach widths are between 130m and 150m	Do Nothing
Beach width lower than 120m	Renourish 1,800m^3 per month until beach width greater than 130m
Beach width greater than 160m and high longshore drift	Recycle 13,500m^3 per month until beach width less than 150m

Using the same time series of wave data used for the 'do nothing' scenario, the following works were simulated in an attempt to manage the beach over a period of 15 years. This period encompassed both recycling and renourishment needs.

Month	Work undertaken
Month 31 to 34	Recycle at 13,500m^3 per month
Month 42 to 43	Recycle at 13,500m^3 per month
Month 75 to 77	Recycle at 13,500m^3 per month
Month 85 to 87	Recycle at 13,500m^3 per month
Month 155 – 180	Renourish at 1,800m^3 per month

The comparison between the 'do nothing' and the beach management approach shows that the beach width can be controlled such that the majority of the time the beach is within the acceptable limits (Figure 15) by recycling a total of 162,000m^3 and renourishing a total of 45,000m^3.

As discussed earlier, the risk of flooding is perhaps easier to manage as remedial works, such as renourishment can be initialised when required or some secondary action such as flood warning initiated as a precaution. Shingle intrusion is more difficult to predict, particularly the optimum time to extract/re-cycle material from the beach. Even with an intensive extraction effort, shingle will still enter the harbour and will require subsequent removal by a dredger. Comparing the results of the 'do nothing' and the beach management strategy shows that by passing is reduced significantly from 23,000m^3 to 8,000m^3 over a period of 15 years (Figure 16).

Regular monitoring will be required to observe the movement of beach material and determine the appropriate action. The impact of carrying out re-cycling and renourishment works on the beach width is plotted in Figure 17 and summarised below:

Percentage of time that Beach Width is at	'Do Nothing'	Observational Beach Management Strategy
120m	8%	14%
130m	33%	37%
150m	18%	13%
160m	10	5%

These results show that carrying out recycling to limit shingle intrusion pushes the then position of the beach landwards. Any work undertaken pre-winter should, therefore, ensure that the flood defence is not compromised.

7. CONCLUSIONS

The study assumed a design profile for the shingle bank comprising a 10m wide crest at +7.5mODN with a 1in 6 leeward slope. Seaward of the crest, the beach sloped at 1 in 3, merged into a 1 in 8 slope. The offshore bathymetry sloped at 1 in 100. The design profile is based upon beach cross sections provided by WDDC in April 1999 in preparation for the physical model.

The conclusions of the study are:

- Modelling of the long-shore and cross-shore processes have indicated that the defined beach width (distance from the base line to the mean water level) should be maintained within the following limits:
  - Extreme Lower Limit = 120m
  - Lower Limit = 130m
  - Upper Limit = 150m
  - Extreme Upper Limit = 160m

- If the do nothing scenario is adopted then there is a high chance that an unacceptable volume of shingle will enter the harbour entrance. This will require excessive dredging to clear the entrance, with attendant environmental and economic consequence.

- The longshore drift can vary in both directions from month to month. It is therefore difficult to identify a set amount to re-cycle or renourish per annum. An observational approach is proposed to reduce the likelihood of beach material by passing East Pier and entering into the harbour. Even with this approach some dredging will still inevitably be required.

- Narrowing the beach by extraction of material in the vicinity of the harbour entrance, in order to minimise the risk of sediment ingress into the harbour, will necessarily increase the risk of local breaching and overtopping of the beach. This is best managed by careful re-use of the re-cycled material. It is suggested that the crest of the berm is locally widened/raised where there is significant room to implement this, rather than re-cycling to a point some distance to the east of the study frontage.

- The results discussed are based on previously occurring wave conditions, and there is no guarantee that similar wave conditions will continue in the future. It is therefore important that the beach is carefully monitored so that robust management decisions are made based on the real time evolution of the beach. As further information is collected, a better understanding of the beach will allow further enhancements to the modelling and decision making process.

- In terms of climate change, the main impact will be any change in storminess (i.e. magnitude of storms and direction) rather than sea level rise as long as the beach is maintained at a width higher than the minimum beach width.

- In terms of large swell events, which are regularly observed in Lyme Bay, the beach will perform satisfactorily as long as it is maintained at a width higher than the minimum beach width. However, after a significant swell event some re-nourishment or re-profiling work is likely to be required.

8. RECOMMENDATIONS

The recommendations of the study are:

1. The beach width should be maintained between 130m and 150m, but should not exceed 160m or be less than 120m. The defined beach width is measured from the model base line.

2. An observational approach to beach management should be adopted with re-cycling and re-nourishment being undertaken when required. Care needs to be taken that to ensure the risk of flooding is not increased to an unacceptable level when re-cycling material.

3. The economic and environmental consequences of re-cycling or allowing material to by-pass East Pier should be carefully assessed in order to determine what the best beach management policy is.

9. REFERENCES

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Tables

Table 1 Prediction of likely number of events causing a failure – present day

Wave Run-up interval Bands		Probability of wave runup occurring within a set band – Beach Width (m)						Probability of Failure	Probability of Failure for a given beach width – Beach Width (m)					
		110	120	125	130	140	150		110	120	125	130	140	150
< -0.5m		0.86494	0.95547	0.98769	0.99429	0.99939	0.99994	0.00	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001
-0.5	0.5	0.01378	0.01124	0.00177	0.00106	0.00012	< 0.00001	**0.00**	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001
0.5	1.5	0.00887	0.00820	0.00136	0.00090	0.00009	< 0.00001	0.00	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001
1.5	2.5	0.00686	0.00595	0.00121	0.00075	0.00008	< 0.00001	0.00	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001
2.5	3.5	0.00552	0.00413	0.00112	0.00058	0.00005	< 0.00001	0.00	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001
3.5	4.5	0.00680	0.00268	0.00111	0.00048	0.00004	< 0.00001	**0.00**	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001
4.5	5.5	0.00771	0.00176	0.00106	0.00038	0.00003	< 0.00001	0.00	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001
5.5	6.5	0.01105	0.00136	0.00090	0.00030	0.00003	< 0.00001	0.00	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001
6.5	7.5	0.00796	0.00121	0.00075	0.00024	0.00002	< 0.00001	0.00	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001
7.5	8.5	0.00901	0.00112	0.00058	0.00019	0.00002	< 0.00001	0.00	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001
8.5	9.5	0.01292	0.00111	0.00048	0.00016	0.00001	< 0.00001	0.00	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001
9.5	10.5	0.01124	0.00106	0.00038	0.00012	0.00001	< 0.00001	**0.00**	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001	< 0.00001
10.5	11.5	0.00820	0.00090	0.00030	0.00009	< 0.00001	< 0.00001	0.05	0.00041	0.00004	0.00001	< 0.00001	< 0.00001	< 0.00001
11.5	12.5	0.00595	0.00075	0.00024	0.00008	< 0.00001	< 0.00001	0.10	0.00059	0.00007	0.00002	< 0.00001	< 0.00001	< 0.00001
12.5	13.5	0.00413	0.00058	0.00019	0.00005	< 0.00001	< 0.00001	0.15	0.00061	0.00008	0.00002	< 0.00001	< 0.00001	< 0.00001
13.5	14.5	0.00268	0.00048	0.00016	0.00004	< 0.00001	< 0.00001	0.20	0.00053	0.00009	0.00003	< 0.00001	< 0.00001	< 0.00001
14.5	15.5	0.00176	0.00038	0.00013	0.00003	< 0.00001	< 0.00001	0.25	0.00044	0.00009	0.00003	< 0.00001	< 0.00001	< 0.00001
15.5	16.5	0.00136	0.00030	0.00009	0.00003	< 0.00001	< 0.00001	**0.30**	0.00041	0.00009	0.00002	< 0.00001	< 0.00001	< 0.00001
16.5	17.5	0.00121	0.00024	0.00008	0.00002	< 0.00001	< 0.00001	0.30	0.00036	0.00007	0.00002	< 0.00001	< 0.00001	< 0.00001
17.5	18.5	0.00112	0.00019	0.00006	0.00002	< 0.00001	< 0.00001	0.30	0.00033	0.00005	0.00001	< 0.00001	< 0.00001	< 0.00001
18.5	19.5	0.00111	0.00016	0.00004	0.00001	< 0.00001	< 0.00001	0.30	0.00033	0.00005	0.00001	< 0.00001	< 0.00001	< 0.00001
19.5	20.5	0.00106	0.00012	0.00003	0.00001	< 0.00001	< 0.00001	0.30	0.00032	0.00003	0.00001	< 0.00001	< 0.00001	< 0.00001
20.5	21.5	0.00090	0.00009	0.00003	< 0.00001	< 0.00001	< 0.00001	0.30	0.00027	0.00002	0.00000	< 0.00001	< 0.00001	< 0.00001
21.5	22.5	0.00075	0.00008	0.00002	< 0.00001	< 0.00001	< 0.00001	**0.30**	0.00022	0.00002	0.00000	< 0.00001	< 0.00001	< 0.00001
22.5	23.5	0.00058	0.00005	0.00002	< 0.00001	< 0.00001	< 0.00001	0.53	0.00031	0.00003	0.00001	< 0.00001	< 0.00001	< 0.00001
23.5	24.5	0.00048	0.00004	0.00001	< 0.00001	< 0.00001	< 0.00001	0.77	0.00037	0.00003	0.00001	< 0.00001	< 0.00001	< 0.00001
> 25m		0.00190	0.00019	0.00005	0.00001	< 0.00001	< 0.00001	**1.00**	0.00190	0.00019	0.00005	< 0.00001	< 0.00001	< 0.00001

	110	120	125	130	140	150
Probability of failure during a single tide for a given beach width	0.00746	0.00103	0.00033	0.00010	0.00001	< 0.00001
Number of likely breach events in a given year	5.2	0.7	0.2	0.1	<0.1	<0.1

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Table 2 Predicted overtopping of East Beach

Return Period (years)	Mean Overtopping Rate	
	+7.5mODN (l/s/m)	+5.0mODN (l/s/m)
0.1	0.0	0.0
0.5	0.0	0.0
1	0.0	0.0
2.5	0.0	0.1
5	0.0	0.2
10	0.0	0.4
100	0.0	2.6
200	0.0	3.7
2000	0.0	12.2

Table 3 Impact of climate change on the likely number of events causing failure

Scenario	Beach Width					
	110	**120**	**125**	**130**	**140**	**150**
Present	5.2	0.7	0.2	<0.1	<0.1	<0.1
5 Years	5.2	0.7	0.2	<0.1	<0.1	<0.1
10 Years	5.5	0.8	0.2	<0.1	<0.1	<0.1
25 Years	5.9	0.8	0.3	<0.1	<0.1	<0.1
50 Years	6.6	0.9	0.3	<0.1	<0.1	<0.1

Table 4 Prediction of likely number of events causing a failure – 5 years sea level rise

Wave Run-up interval Bands		Probability of wave runup occurring within a set band – Beach Width (m) 110	120	125	130	140	150	Probability of Failure	Probability of Failure for a given beach width – Beach Width (m) 110	120	125	130	140	150
> -0.5m		0.86494	0.95443	0.98750	0.99417	0.99937	0.99993	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
-0.5	0.5	0.01378	0.01146	0.00182	0.00108	0.00013	0.00001	**0.00**	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
0.5	1.5	0.00887	0.00844	0.00139	0.00091	0.00009	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
1.5	2.5	0.00686	0.00611	0.00122	0.00077	0.00008	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
2.5	3.5	0.00552	0.00425	0.00112	0.00060	0.00006	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
3.5	4.5	0.00680	0.00279	0.00109	0.00049	0.00004	<0.00001	**0.00**	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
4.5	5.5	0.00771	0.00182	0.00108	0.00039	0.00003	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
5.5	6.5	0.01105	0.00139	0.00091	0.00031	0.00003	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
6.5	7.5	0.00796	0.00122	0.00077	0.00024	0.00002	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
7.5	8.5	0.00901	0.00112	0.00060	0.00019	0.00002	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
8.5	9.5	0.01292	0.00109	0.00049	0.00017	0.00001	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
9.5	10.5	0.01124	0.00108	0.00039	0.00013	0.00001	<0.00001	**0.00**	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
10.5	11.5	0.00820	0.00091	0.00031	0.00010	<0.00001	<0.00001	0.05	0.000410	0.000045	0.000015	0.000005	<0.00001	<0.00001
11.5	12.5	0.00595	0.00077	0.00024	0.00008	<0.00001	<0.00001	0.10	0.000595	0.000077	0.000024	0.000008	<0.00001	<0.00001
12.5	13.5	0.00413	0.00060	0.00019	0.00006	<0.00001	<0.00001	0.15	0.000619	0.000090	0.000029	0.000009	0.000001	<0.00001
13.5	14.5	0.00268	0.00049	0.00017	0.00004	<0.00001	<0.00001	0.20	0.000537	0.000098	0.000034	0.000009	0.000001	<0.00001
14.5	15.5	0.00176	0.00039	0.00013	0.00003	<0.00001	<0.00001	0.25	0.000442	0.000097	0.000033	0.000009	<0.00001	<0.00001
15.5	16.5	0.00136	0.00031	0.00010	0.00003	<0.00001	<0.00001	**0.30**	0.000410	0.000094	0.000030	0.000009	<0.00001	<0.00001
16.5	17.5	0.00121	0.00024	0.00008	0.00002	<0.00001	<0.00001	0.30	0.000364	0.000074	0.000024	0.000008	<0.00001	<0.00001
17.5	18.5	0.00112	0.00019	0.00006	0.00002	<0.00001	<0.00001	0.30	0.000336	0.000059	0.000019	0.000006	<0.00001	<0.00001
18.5	19.5	0.00111	0.00017	0.00004	0.00001	<0.00001	<0.00001	0.30	0.000334	0.000051	0.000013	0.000005	<0.00001	<0.00001
19.5	20.5	0.00106	0.00013	0.00003	0.00001	<0.00001	<0.00001	0.30	0.000319	0.000039	0.000011	0.000003	<0.00001	<0.00001
20.5	21.5	0.00090	0.00009	0.00003	<0.00001	<0.00001	<0.00001	0.30	0.000270	0.000029	0.000009	0.000002	<0.00001	<0.00001
21.5	22.5	0.00075	0.00008	0.00002	<0.00001	<0.00001	<0.00001	**0.30**	0.000226	0.000024	0.000008	0.000002	<0.00001	<0.00001
22.5	23.5	0.00058	0.00006	0.00002	<0.00001	<0.00001	<0.00001	0.53	0.000313	0.000034	0.000011	0.000003	<0.00001	<0.00001
23.5	24.5	0.00048	0.00004	0.00001	<0.00001	<0.00001	<0.00001	0.77	0.000374	0.000036	0.000013	0.000004	<0.00001	<0.00001
> 25m		0.00190	0.00019	0.00006	<0.00001	<0.00001	<0.00001	1.00	0.001906	0.000199	0.000060	0.000018	0.000001	<0.00001

	110	120	125	130	140	150
Probability of failure during a single tide for a given beach width	0.007461	0.001054	0.000341	0.000107	0.000010	0.000001
Number of likely breach events in a given year	5.2	0.7	0.2	<0.1	<0.1	<0.1

Table 5 Prediction of likely number of events causing a failure – 10 years sea level rise

Wave Run-up interval Bands		Probability of wave runup occurring within a set band – Beach Width (m) 110	120	125	130	140	150
> -0.5m		0.86371	0.95338	0.98730	0.99406	0.99935	0.99993
-0.5	0.5	0.01261	0.01165	0.00187	0.00108	0.00013	0.00001
0.5	1.5	0.01049	0.00867	0.00141	0.00093	0.00010	<0.00001
1.5	2.5	0.00665	0.00626	0.00123	0.00078	0.00008	<0.00001
2.5	3.5	0.00595	0.00441	0.00113	0.00061	0.00006	<0.00001
3.5	4.5	0.00647	0.00291	0.00109	0.00050	0.00004	<0.00001
4.5	5.5	0.00712	0.00187	0.00108	0.00039	0.00004	<0.00001
5.5	6.5	0.01096	0.00141	0.00093	0.00031	0.00003	<0.00001
6.5	7.5	0.00829	0.00123	0.00078	0.00025	0.00002	<0.00001
7.5	8.5	0.00848	0.00113	0.00061	0.00020	0.00002	<0.00001
8.5	9.5	0.01259	0.00109	0.00050	0.00017	0.00001	<0.00001
9.5	10.5	0.01165	0.00108	0.00039	0.00013	0.00001	<0.00001
10.5	11.5	0.00867	0.00093	0.00031	0.00010	<0.00001	<0.00001
11.5	12.5	0.00626	0.00078	0.00025	0.00008	<0.00001	<0.00001
12.5	13.5	0.00441	0.00061	0.00020	0.00006	<0.00001	<0.00001
13.5	14.5	0.00291	0.00050	0.00017	0.00004	<0.00001	<0.00001
14.5	15.5	0.00187	0.00039	0.00013	0.00004	<0.00001	<0.00001
15.5	16.5	0.00141	0.00031	0.00010	0.00003	<0.00001	<0.00001
16.5	17.5	0.00123	0.00025	0.00008	0.00002	<0.00001	<0.00001
17.5	18.5	0.00113	0.00020	0.00006	0.00002	<0.00001	<0.00001
18.5	19.5	0.00109	0.00017	0.00004	0.00001	<0.00001	<0.00001
19.5	20.5	0.00108	0.00013	0.00004	0.00001	<0.00001	<0.00001
20.5	21.5	0.00093	0.00010	0.00003	<0.00001	<0.00001	<0.00001
21.5	22.5	0.00078	0.00008	0.00002	<0.00001	<0.00001	<0.00001
22.5	23.5	0.00061	0.00006	0.00002	<0.00001	<0.00001	<0.00001
23.5	24.5	0.00050	0.00004	0.00001	<0.00001	<0.00001	<0.00001
> 25m		0.00199	0.00020	0.00006	0.00001	<0.00001	<0.00001

Probability of Failure	Probability of Failure for a given beach width – Beach Width (m) 110	120	125	130	140	150
0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
0.05	0.000433	0.000046	0.000015	0.000005	<0.00001	<0.00001
0.10	0.000626	0.000078	0.000025	0.000008	<0.00001	<0.00001
0.15	0.000662	0.000092	0.000030	0.000010	0.000001	<0.00001
0.20	0.000582	0.000100	0.000035	0.000009	0.000001	<0.00001
0.25	0.000468	0.000099	0.000034	0.000010	<0.00001	<0.00001
0.30	0.000424	0.000095	0.000030	0.000009	<0.00001	<0.00001
0.30	0.000371	0.000076	0.000024	0.000008	<0.00001	<0.00001
0.30	0.000339	0.000060	0.000020	0.000006	0.000001	<0.00001
0.30	0.000329	0.000053	0.000014	0.000005	<0.00001	<0.00001
0.30	0.000326	0.000041	0.000012	0.000003	<0.00001	<0.00001
0.30	0.000279	0.000030	0.000009	0.000002	<0.00001	<0.00001
0.30	0.000236	0.000024	0.000008	0.000002	<0.00001	<0.00001
0.53	0.000330	0.000035	0.000012	0.000003	<0.00001	<0.00001
0.77	0.000384	0.000037	0.000013	0.000004	<0.00001	<0.00001
1.00	0.001995	0.000203	0.000062	0.000019	0.000001	<0.00001

	110	120	125	130	140	150
Probability of failure during a single tide for a given beach width	0.007792	0.001078	0.000351	0.000110	0.000010	0.000001
Number of likely breach events in a given year	5.5	0.8	0.2	<0.1	<0.1	<0.1

Table 6 Prediction of likely number of events causing a failure – 25 years sea level rise

Wave Run-up interval Bands		Probability of wave runup occurring within a set band – Beach Width (m) 110	120	125	130	140	150	Probability of Failure	Probability of Failure for a given beach width – Beach Width (m) 110	120	125	130	140	150
> -0.5m		0.86113	0.95021	0.98664	0.99374	0.99930	0.99993	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
-0.5	0.5	0.01246	0.01212	0.00209	0.00110	0.00014	0.00001	**0.00**	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
0.5	1.5	0.01092	0.00941	0.00149	0.00097	0.00011	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
1.5	2.5	0.00759	0.00674	0.00127	0.00082	0.00008	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
2.5	3.5	0.00653	0.00489	0.00114	0.00066	0.00007	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
3.5	4.5	0.00472	0.00324	0.00108	0.00053	0.00005	<0.00001	**0.00**	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
4.5	5.5	0.00744	0.00208	0.00110	0.00043	0.00004	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
5.5	6.5	0.01015	0.00149	0.00097	0.00033	0.00003	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
6.5	7.5	0.00935	0.00127	0.00082	0.00028	0.00003	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
7.5	8.5	0.00809	0.00114	0.00066	0.00020	0.00002	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
8.5	9.5	0.01178	0.00108	0.00053	0.00018	0.00001	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
9.5	10.5	0.01212	0.00110	0.00043	0.00014	0.00001	<0.00001	**0.00**	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
10.5	11.5	0.00941	0.00097	0.00033	0.00011	<0.00001	<0.00001	0.05	0.000470	0.000048	0.000016	0.000005	<0.00001	<0.00001
11.5	12.5	0.00674	0.00083	0.00028	0.00008	<0.00001	<0.00001	0.10	0.000674	0.000083	0.000028	0.000008	<0.00001	<0.00001
12.5	13.5	0.00489	0.00065	0.00020	0.00007	<0.00001	<0.00001	0.15	0.000734	0.000098	0.000031	0.000011	0.000001	<0.00001
13.5	14.5	0.00324	0.00052	0.00018	0.00005	<0.00001	<0.00001	0.20	0.000649	0.000105	0.000037	0.000010	0.000001	<0.00001
14.5	15.5	0.00208	0.00043	0.00014	0.00003	<0.00001	<0.00001	0.25	0.000522	0.000108	0.000037	0.000009	0.000001	<0.00001
15.5	16.5	0.00149	0.00033	0.00011	0.00003	<0.00001	<0.00001	**0.30**	0.000448	0.000101	0.000033	0.000010	<0.00001	<0.00001
16.5	17.5	0.00127	0.00028	0.00008	0.00003	<0.00001	<0.00001	0.30	0.000381	0.000084	0.000026	0.000009	<0.00001	<0.00001
17.5	18.5	0.00114	0.00020	0.00007	0.00002	<0.00001	<0.00001	0.30	0.000344	0.000062	0.000022	0.000007	0.000001	<0.00001
18.5	19.5	0.00108	0.00018	0.00005	0.00001	<0.00001	<0.00001	0.30	0.000326	0.000055	0.000016	0.000005	<0.00001	<0.00001
19.5	20.5	0.00110	0.00014	0.00003	0.00001	<0.00001	<0.00001	0.30	0.000332	0.000044	0.000011	0.000004	<0.00001	<0.00001
20.5	21.5	0.00097	0.00011	0.00003	<0.00001	<0.00001	<0.00001	0.30	0.000293	0.000033	0.000010	0.000002	<0.00001	<0.00001
21.5	22.5	0.00083	0.00008	0.00003	<0.00001	<0.00001	<0.00001	**0.30**	0.000249	0.000026	0.000009	0.000002	<0.00001	<0.00001
22.5	23.5	0.00065	0.00007	0.00002	<0.00001	<0.00001	<0.00001	0.53	0.000351	0.000039	0.000012	0.000004	<0.00001	<0.00001
23.5	24.5	0.00052	0.00005	0.00001	<0.00001	<0.00001	<0.00001	0.77	0.000406	0.000041	0.000013	0.000004	<0.00001	<0.00001
> 25m		0.00214	0.00021	0.00006	0.00002	<0.00001	<0.00001	1.00	0.002144	0.000216	0.000069	0.000021	0.000001	<0.00001

	110	120	125	130	140	150
Probability of failure *during a single tide* for a given beach width	0.008330	0.001152	0.000377	0.000118	0.000011	0.000001
Number of likely breach events in a given year	5.9	0.8	0.3	<0.1	<0.1	<0.1

Table 7 Prediction of likely number of events causing a failure – 50 years sea level rise

Wave Run-up interval Bands		Probability of wave runup occurring within a set band – Beach Width (m) 110	120	125	130	140	150	Probability of Failure	Probability of Failure for a given beach width – Beach Width (m) 110	120	125	130	140	150
< -0.5m		0.85779	0.94466	0.98533	0.99322	0.99920	0.99992	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
-0.5	0.5	0.01083	0.01275	0.00255	0.00109	0.00017	0.00001	**0.00**	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
0.5	1.5	0.01214	0.01063	0.00169	0.00105	0.00012	0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
1.5	2.5	0.00916	0.00774	0.00134	0.00088	0.00009	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
2.5	3.5	0.00628	0.00563	0.00119	0.00074	0.00008	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
3.5	4.5	0.00485	0.00391	0.00110	0.00058	0.00006	<0.00001	**0.00**	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
4.5	5.5	0.00718	0.00254	0.00109	0.00048	0.00004	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
5.5	6.5	0.00755	0.00169	0.00105	0.00037	0.00003	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
6.5	7.5	0.01150	0.00134	0.00088	0.00030	0.00003	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
7.5	8.5	0.00776	0.00119	0.00074	0.00024	0.00002	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
8.5	9.5	0.00957	0.00110	0.00058	0.00019	0.00002	<0.00001	0.00	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
9.5	10.5	0.01275	0.00109	0.00048	0.00017	0.00001	<0.00001	**0.00**	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001	<0.00001
10.5	11.5	0.01063	0.00106	0.00037	0.00013	0.00001	<0.00001	0.05	0.000531	0.000053	0.000018	0.000006	<0.00001	<0.00001
11.5	12.5	0.00774	0.00088	0.00030	0.00009	<0.00001	<0.00001	0.10	0.000774	0.000088	0.000030	0.000009	<0.00001	<0.00001
12.5	13.5	0.00563	0.00074	0.00024	0.00008	<0.00001	<0.00001	0.15	0.000845	0.000111	0.000036	0.000012	<0.00001	<0.00001
13.5	14.5	0.00391	0.00058	0.00019	0.00006	<0.00001	<0.00001	0.20	0.000782	0.000116	0.000039	0.000012	0.000001	<0.00001
14.5	15.5	0.00254	0.00048	0.00017	0.00004	<0.00001	<0.00001	0.25	0.000637	0.000120	0.000042	0.000011	0.000001	<0.00001
15.5	16.5	0.00169	0.00038	0.00013	0.00003	<0.00001	<0.00001	**0.30**	0.000508	0.000114	0.000039	0.000011	<0.00001	<0.00001
16.5	17.5	0.00134	0.00030	0.00009	0.00003	<0.00001	<0.00001	0.30	0.000402	0.000092	0.000029	0.000009	<0.00001	<0.00001
17.5	18.5	0.00119	0.00024	0.00008	0.00002	<0.00001	<0.00001	0.30	0.000358	0.000073	0.000024	0.000008	<0.00001	<0.00001
18.5	19.5	0.00110	0.00019	0.00006	0.00002	<0.00001	<0.00001	0.30	0.000330	0.000058	0.000019	0.000006	0.000001	<0.00001
19.5	20.5	0.00109	0.00017	0.00004	0.00001	<0.00001	<0.00001	0.30	0.000328	0.000051	0.000014	0.000005	<0.00001	<0.00001
20.5	21.5	0.00106	0.00012	0.00003	0.00001	<0.00001	<0.00001	0.30	0.000318	0.000038	0.000011	0.000003	<0.00001	<0.00001
21.5	22.5	0.00088	0.00009	0.00003	0.00000	<0.00001	<0.00001	**0.30**	0.000264	0.000029	0.000009	0.000002	<0.00001	<0.00001
22.5	23.5	0.00074	0.00008	0.00002	0.00000	<0.00001	<0.00001	0.53	0.000396	0.000043	0.000015	0.000004	<0.00001	<0.00001
23.5	24.5	0.00058	0.00006	0.00002	0.00000	<0.00001	<0.00001	0.77	0.000447	0.000049	0.000017	0.000005	<0.00001	<0.00001
> 25m		0.00240	0.00024	0.00007	0.00002	<0.00001	<0.00001	1.00	0.002404	0.000247	0.000079	0.000024	0.000002	<0.00001

	110	120	125	130	140	150
Probability of failure during a single tide for a given beach width	0.009331	0.001288	0.000427	0.000135	0.000013	0.000001
Number of likely breach events in a given year	6.6	0.9	0.3	<0.1	<0.1	<0.1

Figures

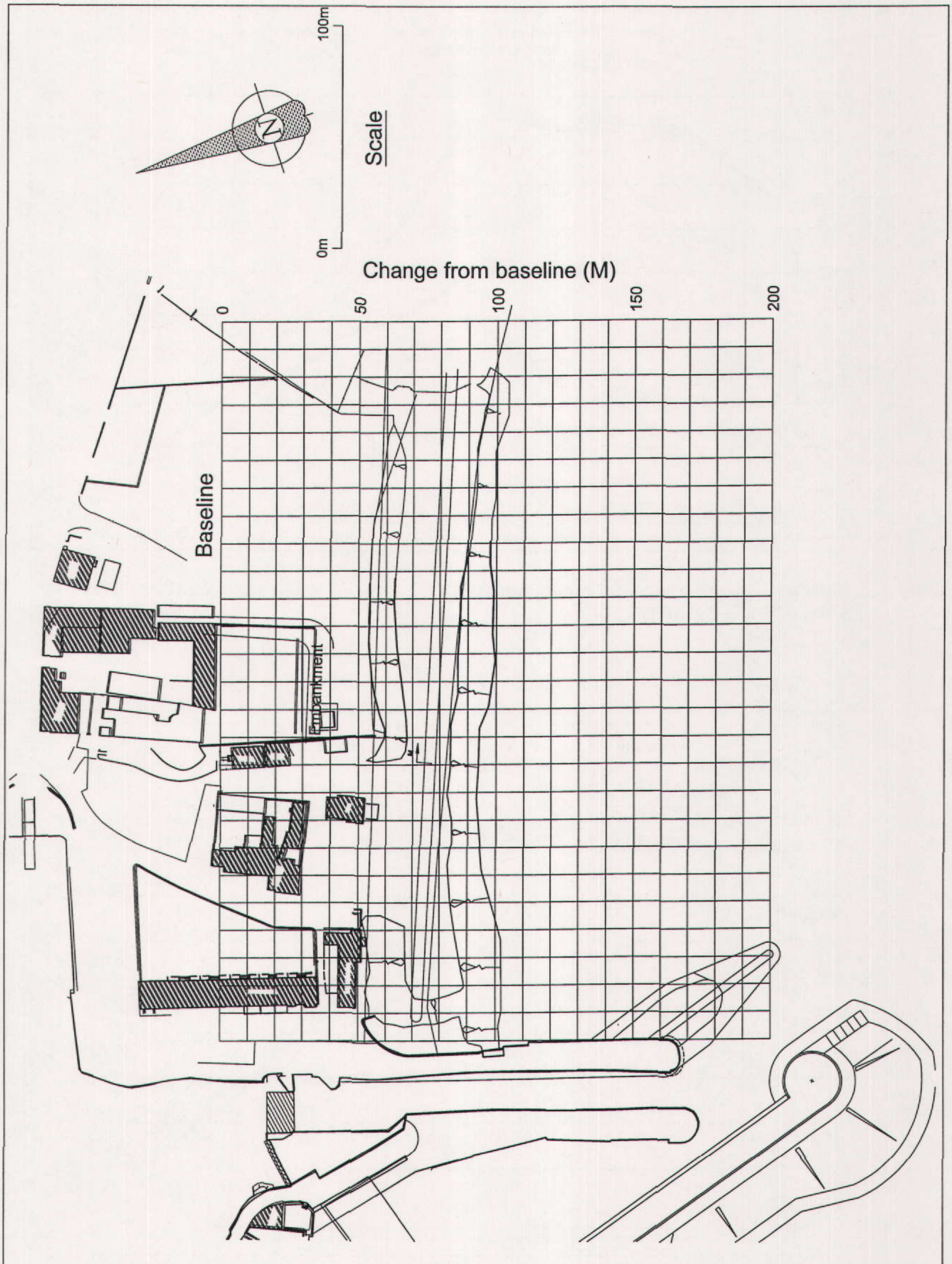

Figure 1 Plan of East Beach

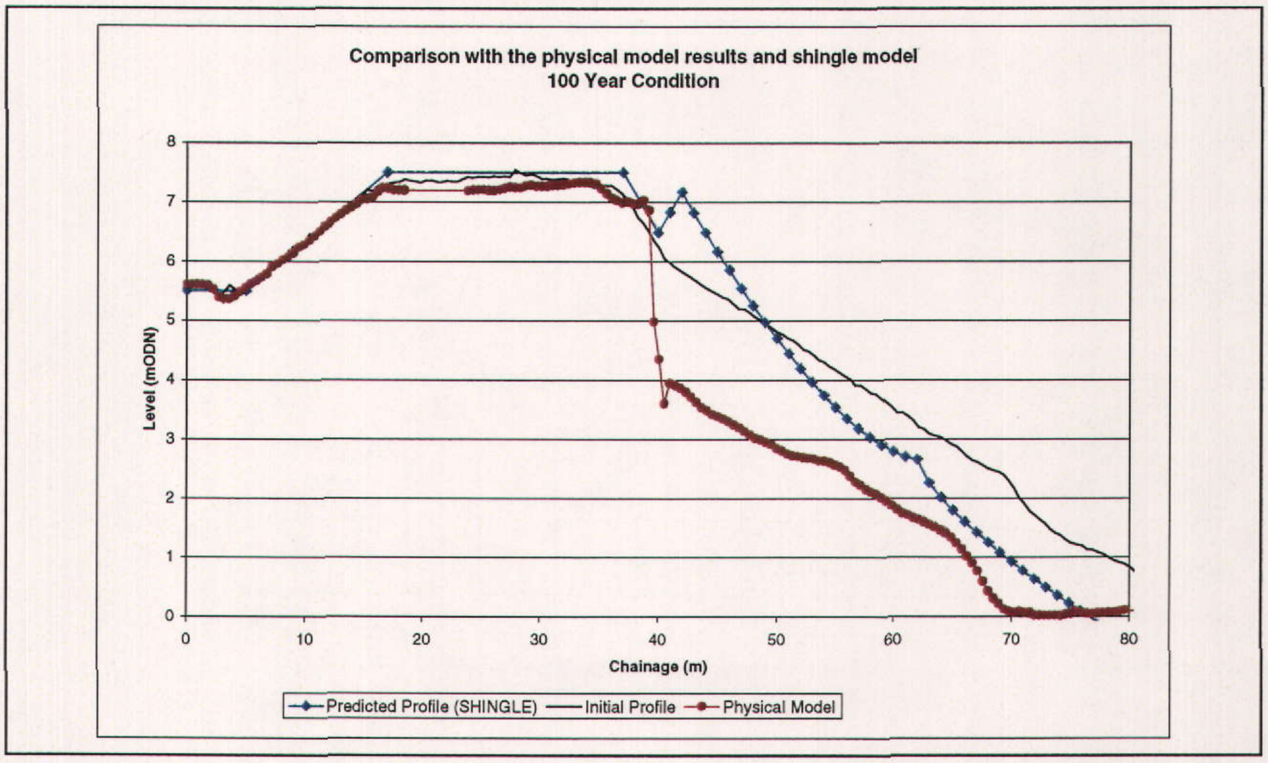

Figure 2 **Comparison of the physical model and SHINGLE model – 100 year condition (Hs = 5.0m, HWL = 2.6mODN)**

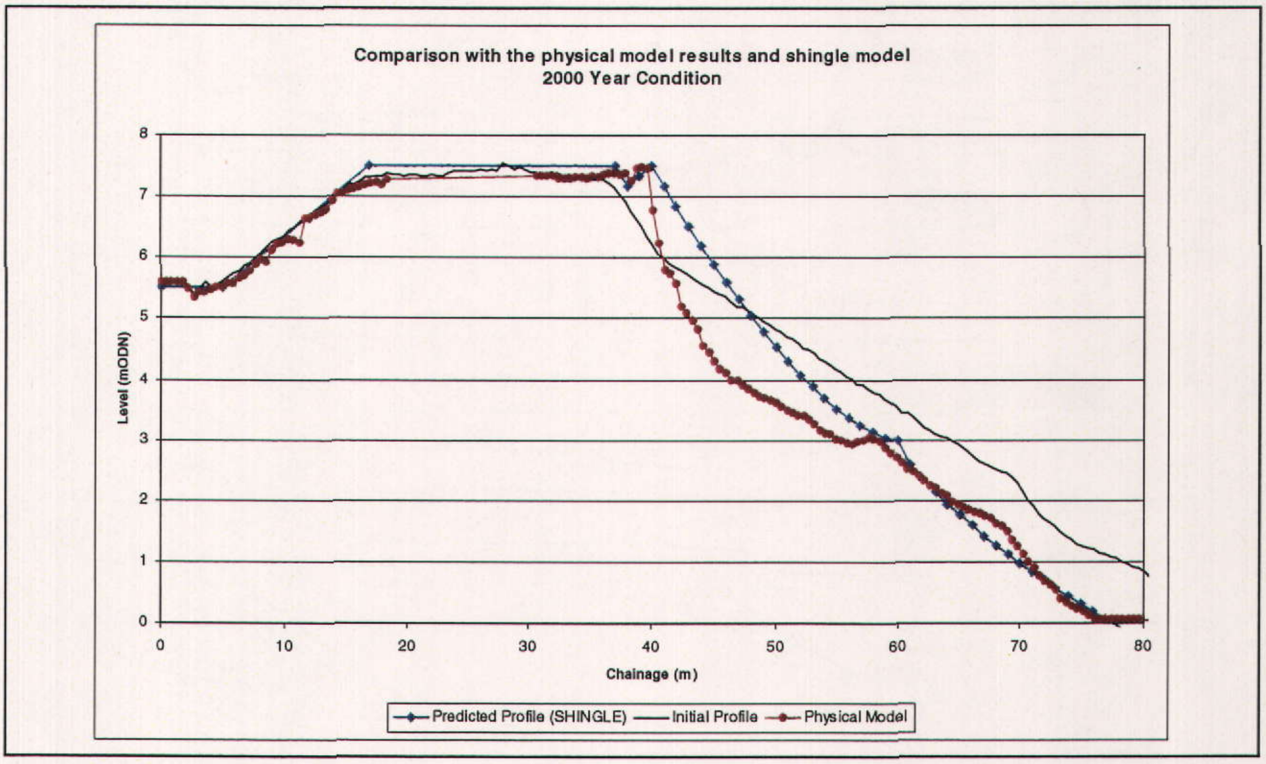

Figure 3 **Comparison of the physical model and SHINGLE model – 2000 year condition (Hs = 5.0m, HWL = 3.0mODN)**

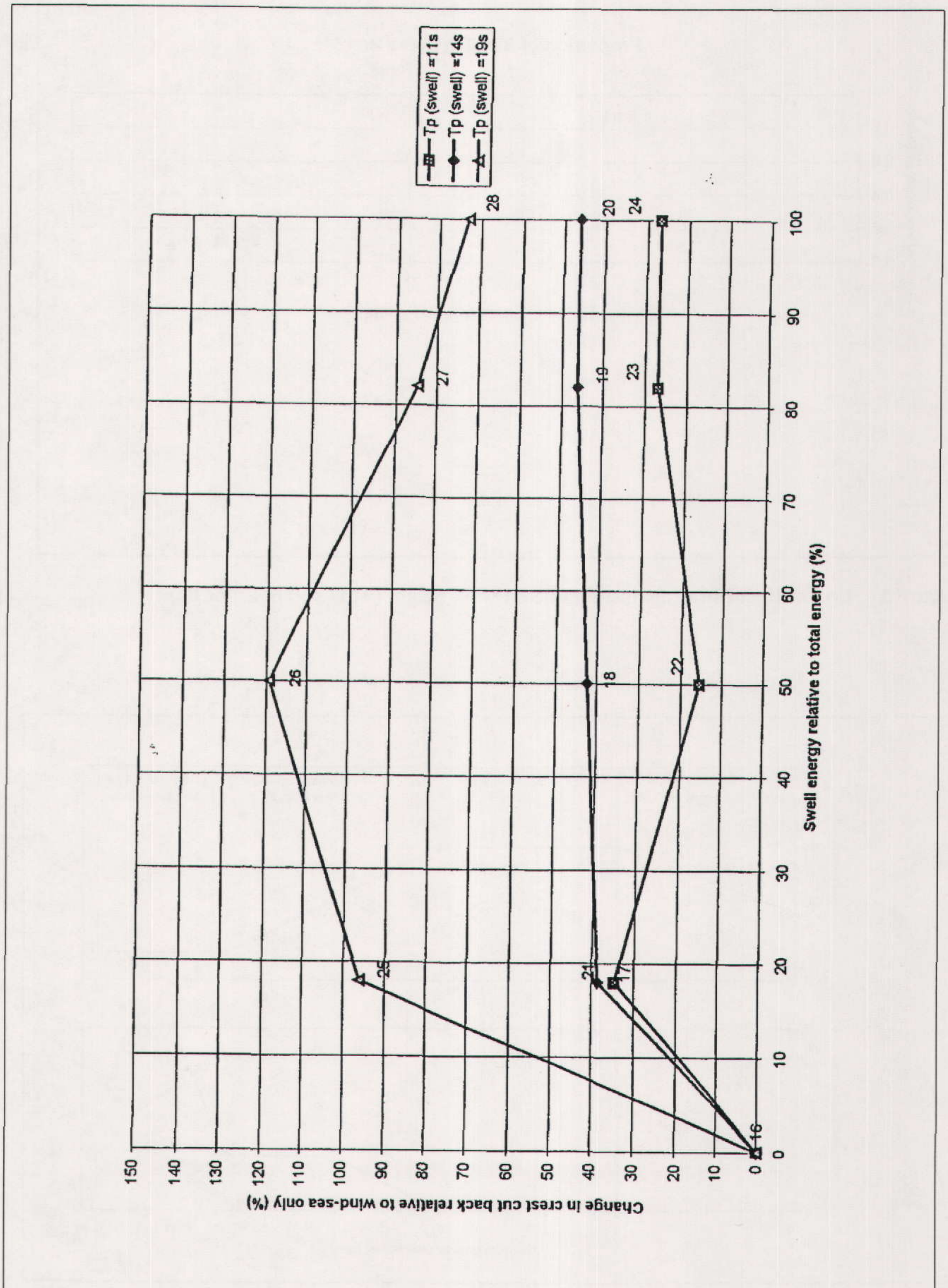

Figure 4 Wave run up in bi-modal seas (Hawkes et al, 1997)

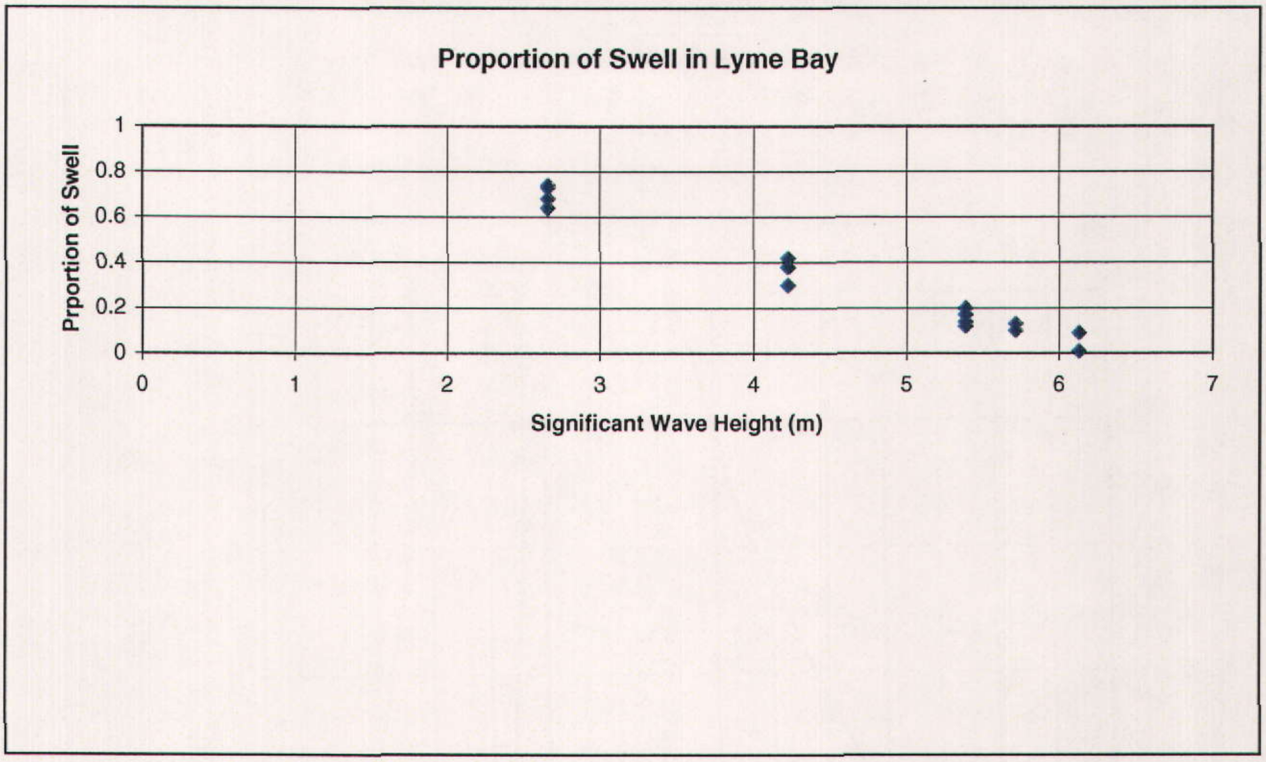

Figure 5 Proportion of swell against significant wave height (wind sea) in Lyme Bay

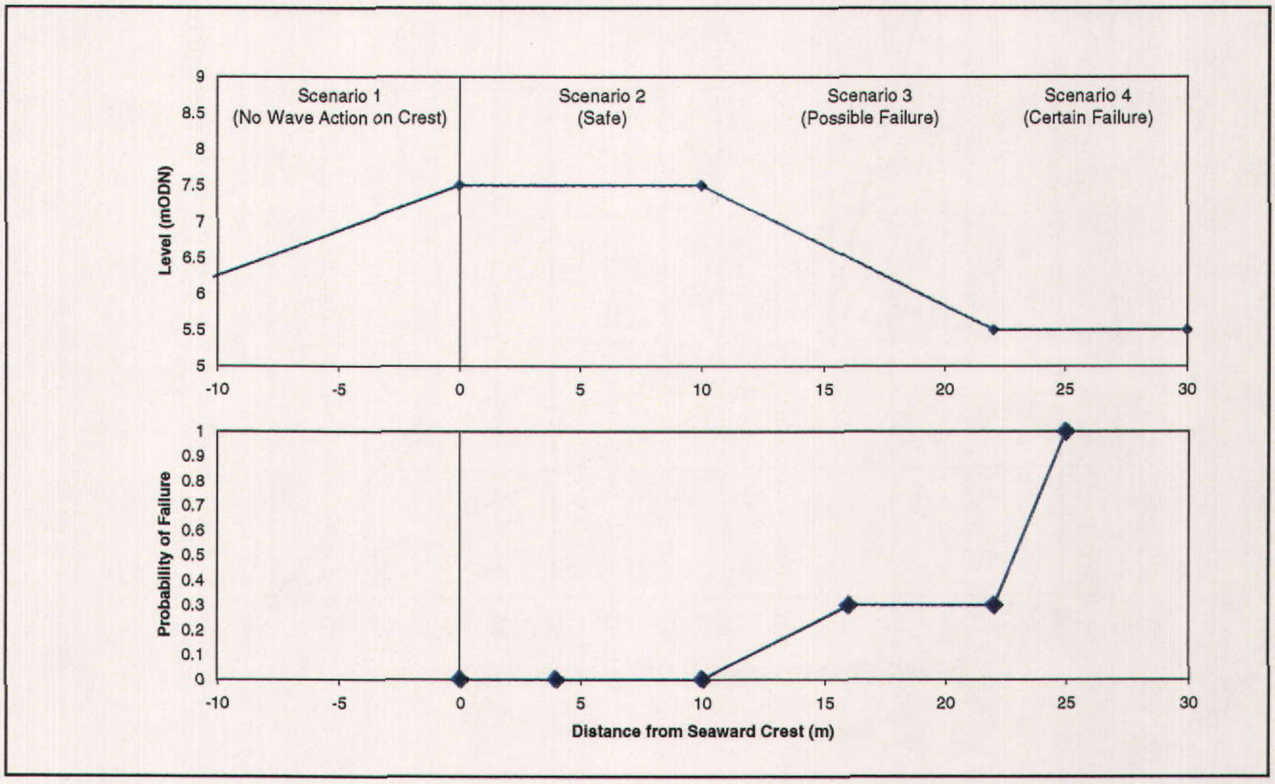

Figure 6 Critical failure scenarios

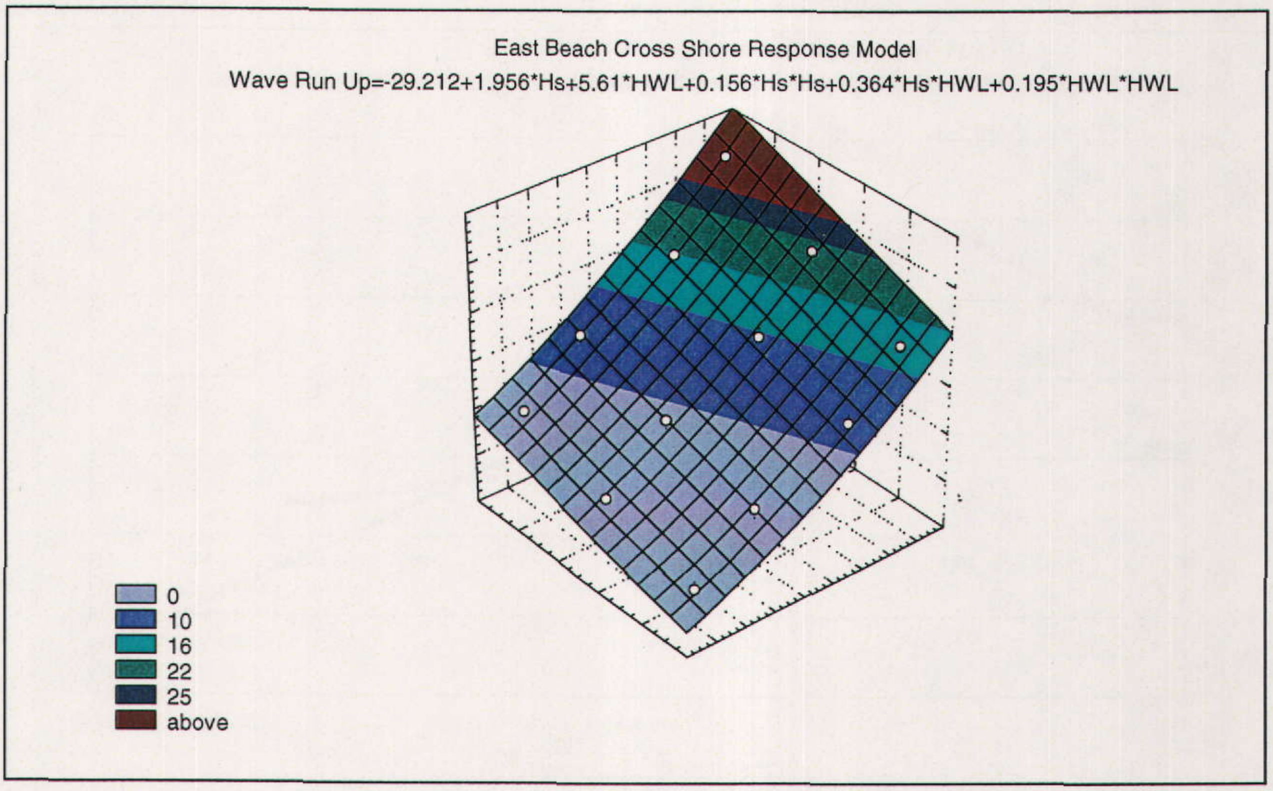

Figure 7 East Beach cross shore response model (beach width = 130m)

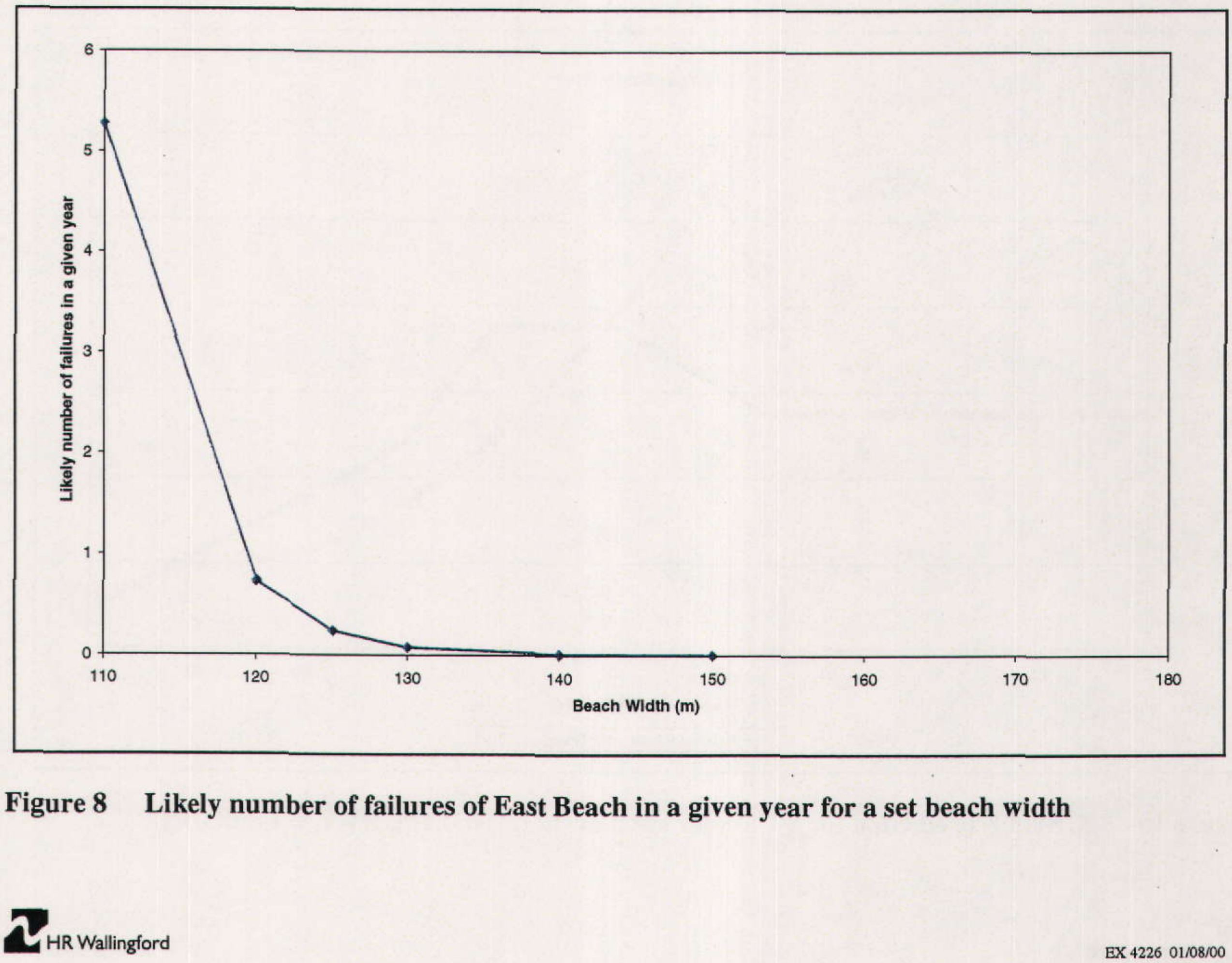

Figure 8 Likely number of failures of East Beach in a given year for a set beach width

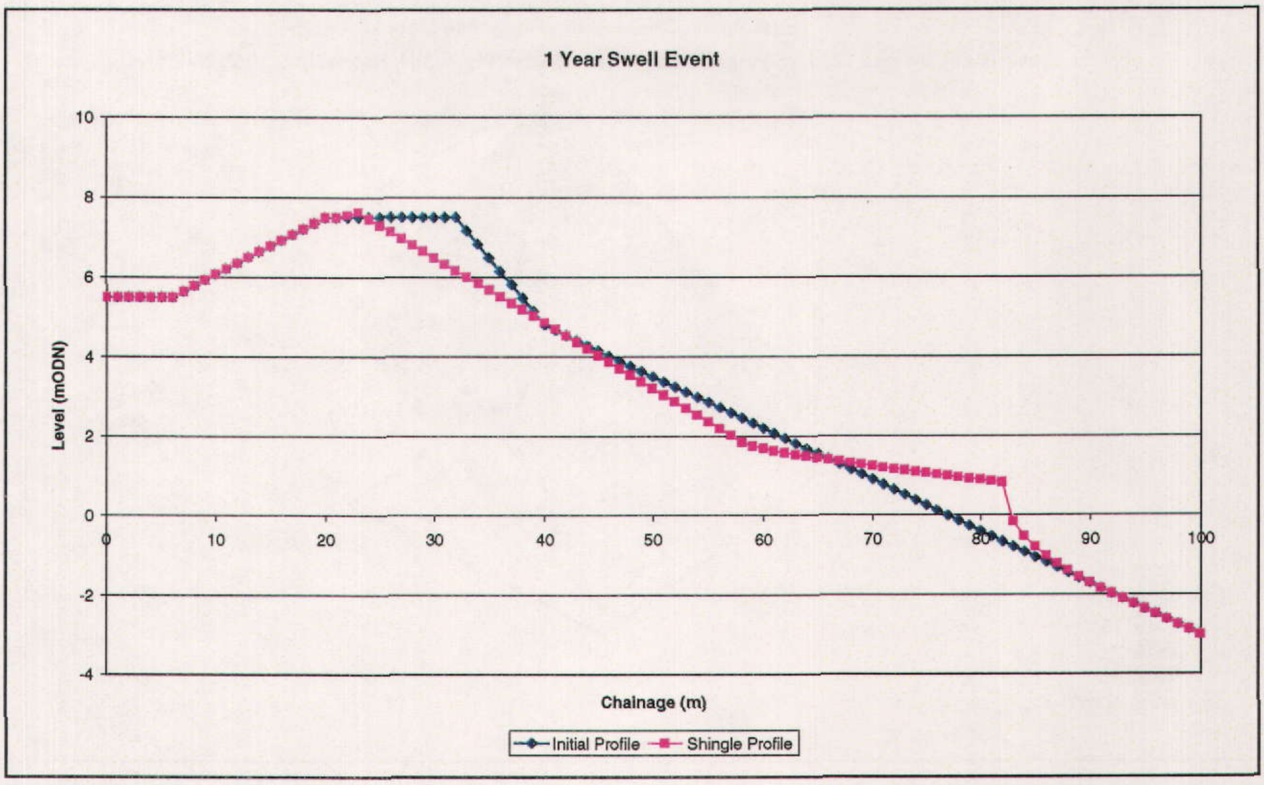

Figure 9 SHINGLE prediction for a 1 year swell event (Hs = 2.3m, HWL = 1.8mODN)

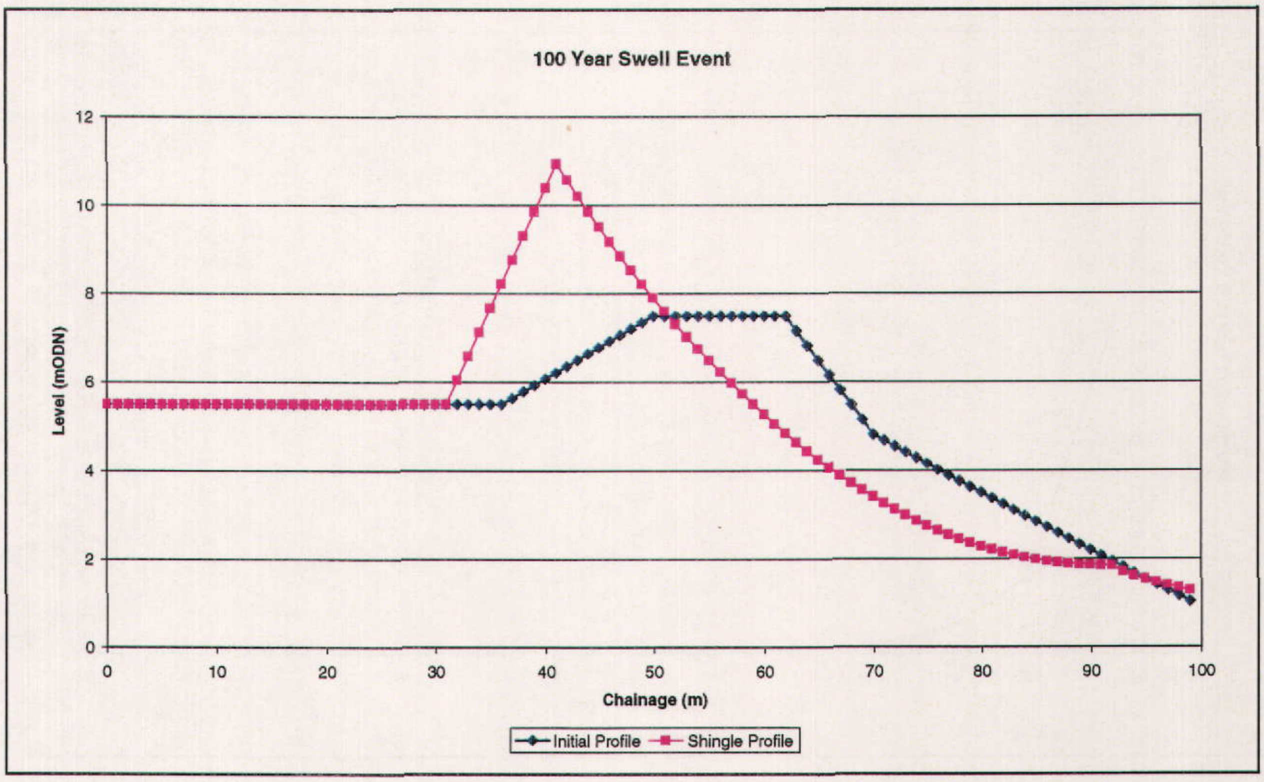

Figure 10 SHINGLE prediction for a 100 year swell event (Hs = 4.1m, HWL = 1.8mODN)

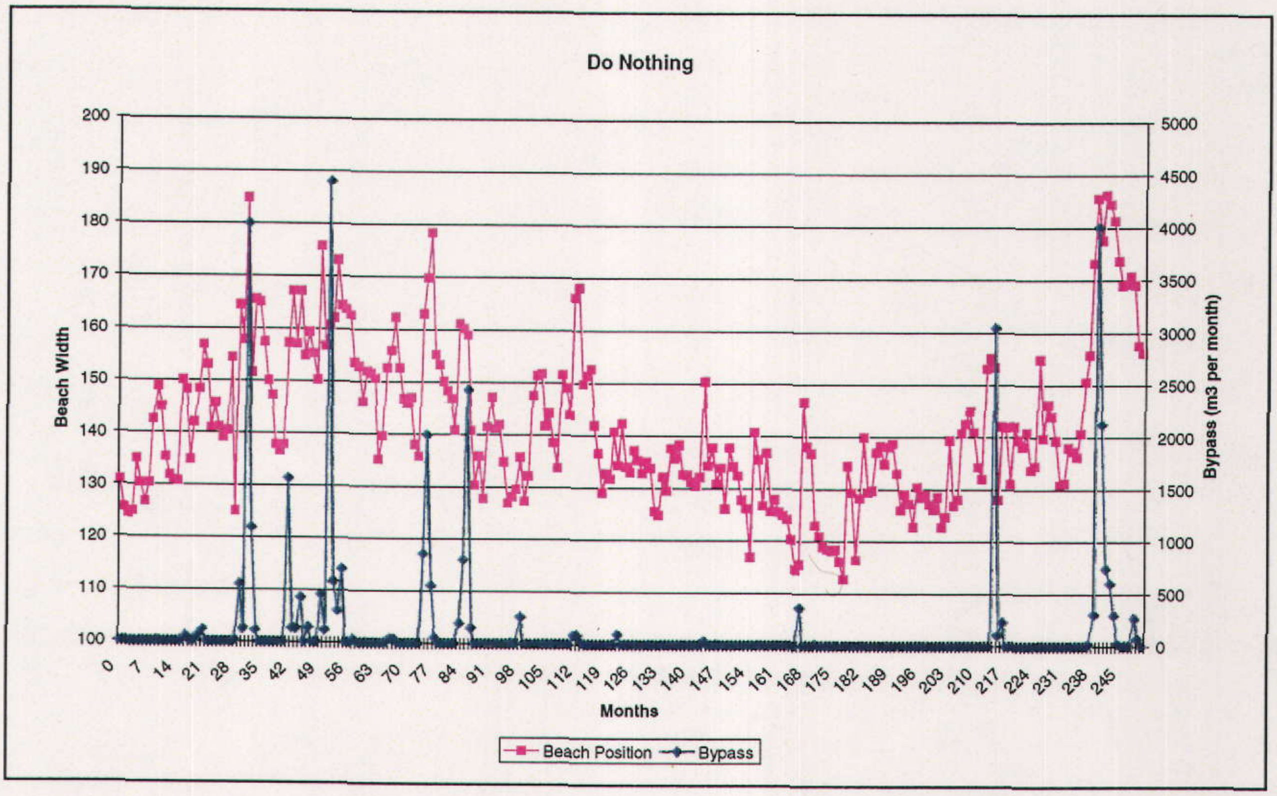

Figure 11 Time series of beach width and bypassing of East Pier Extension – do nothing scenario

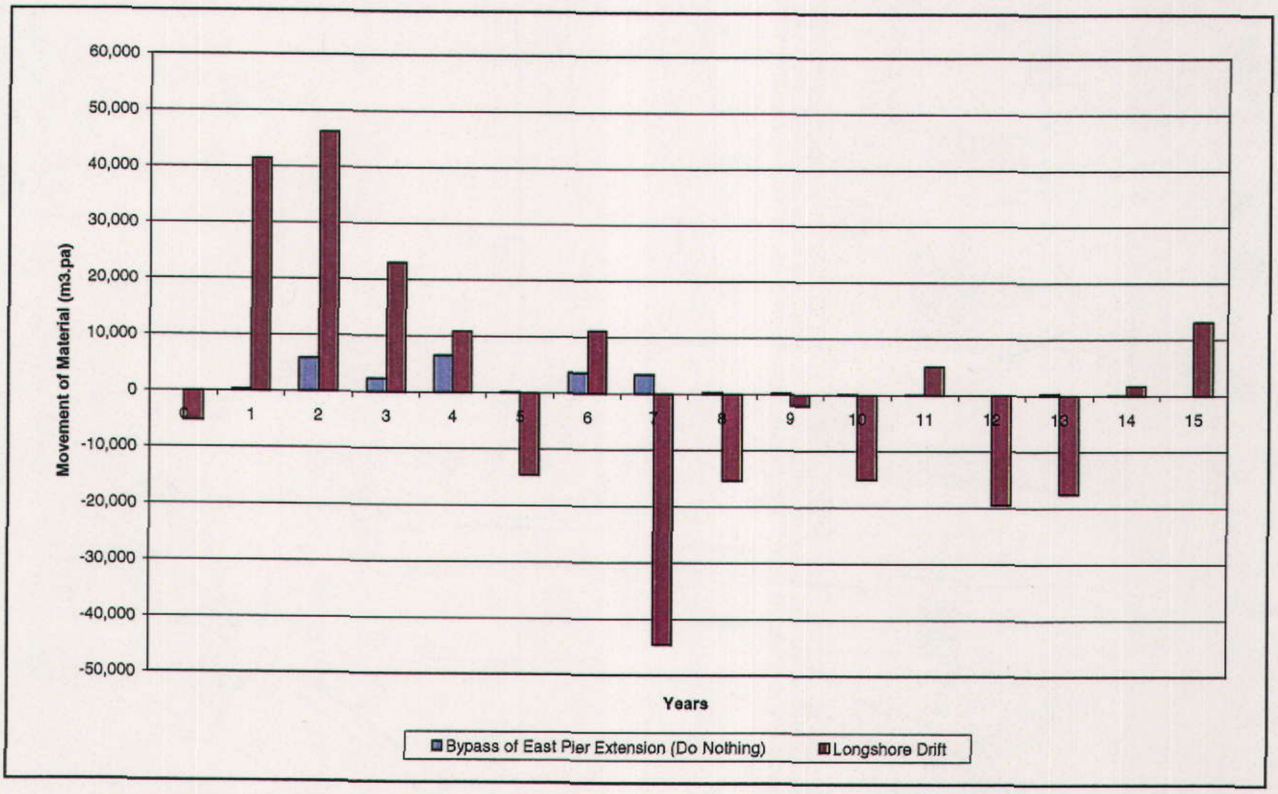

Figure 12 Comparison between the bypass of East Pier Extension and longshore drift

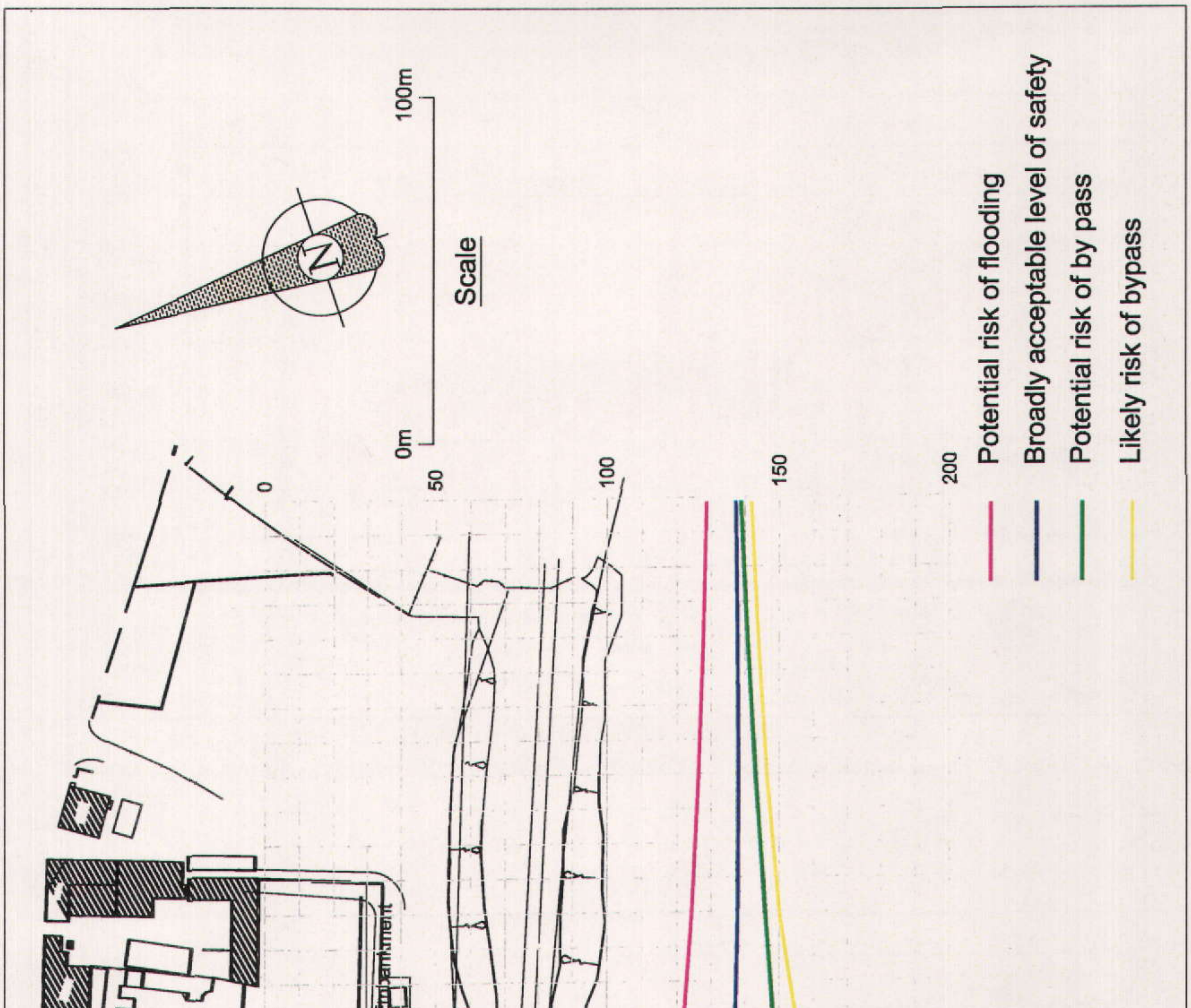

Figure 13 Summary of possible beach plan shapes and their implications

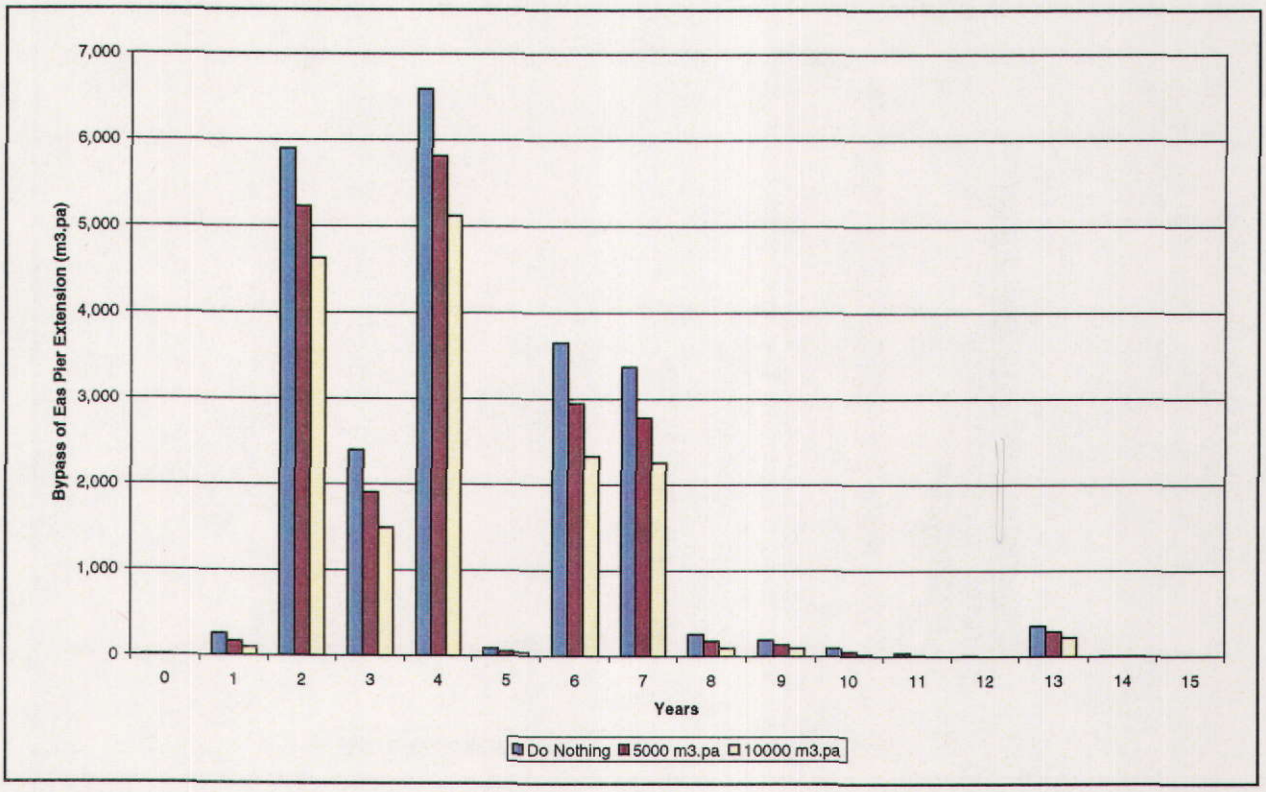

Figure 14 Comparison of simple beach management strategies on controlling bypass

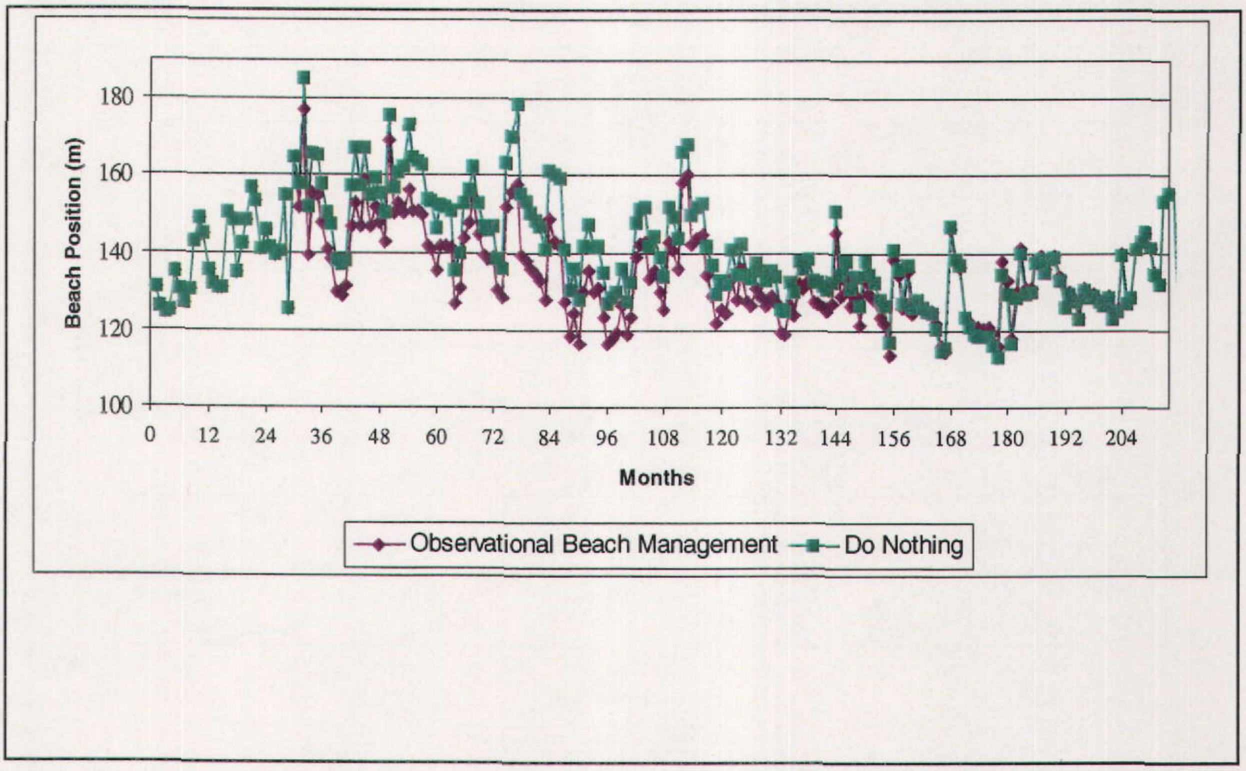

Figure 15 Comparison of beach widths between the do nothing scenario and the beach management strategy

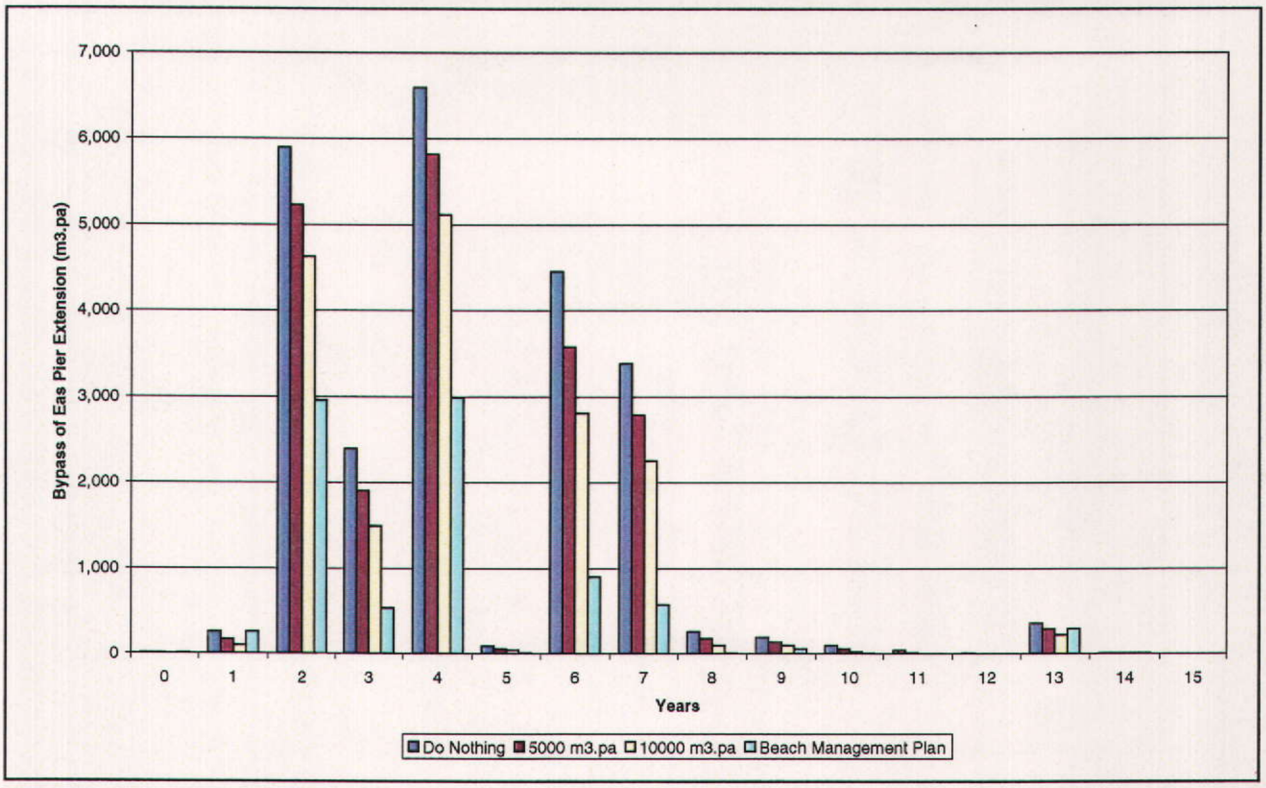

Figure 16 Impact of beach management strategy on controlling bypass

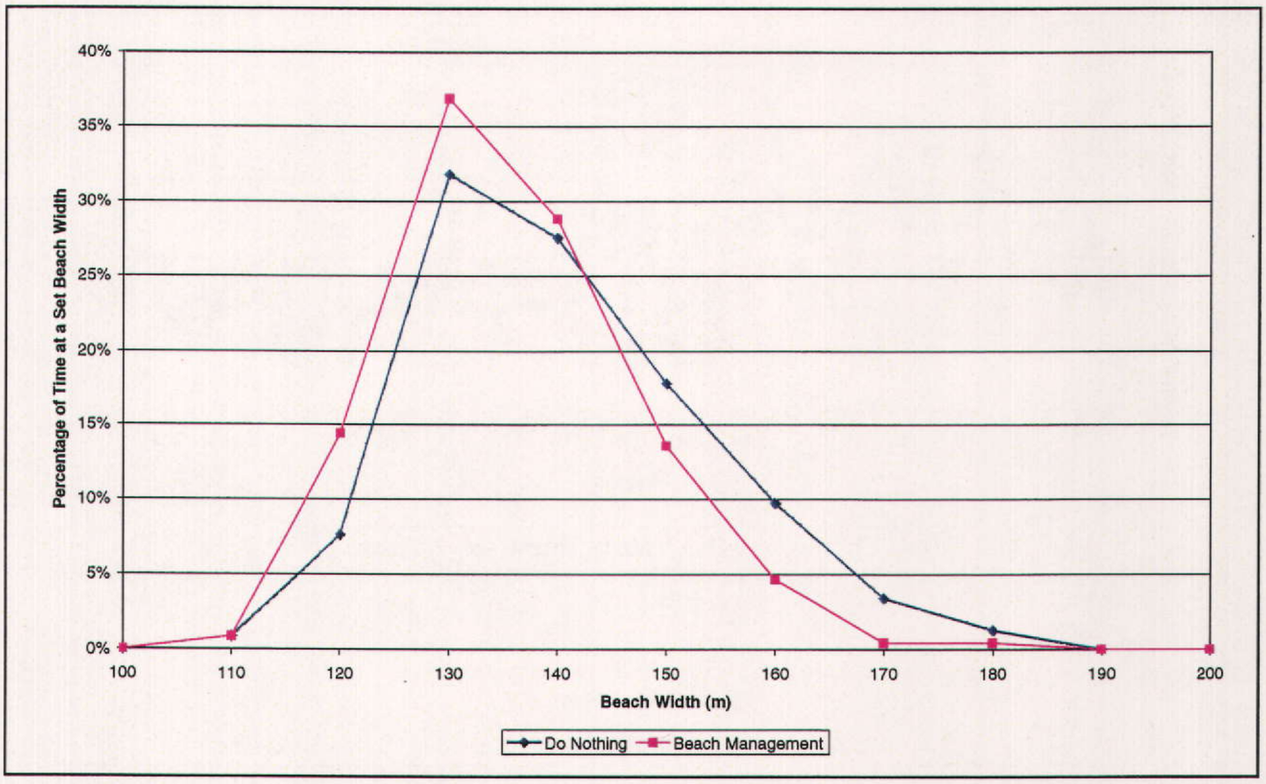

Figure 17 Frequency distribution of beach width