6 PEMBROKE BAY

6.1 Introduction

Pembroke Bay is formed as a wide northeast opening bay between the two hard rock headlands of Fort Pembroke, to the west, and Fort Le Marchant, on the eastern side (Figure 6.1). The bay is backed by the L’Ancresse Common, which includes one of the main golf courses on Guernsey, as well as being an area important for its heritage and historical landscape and, together with the bay, beach and natural environment, is an important amenity and tourism destination.

Figure 6.1. Location Plan of Pembroke Bay

The backshore of the bay is protected by various sections of old military tank defences, extending in effect around the whole soft hinterland. In several areas these old defences are in very poor condition and have required extensive maintenance in recent years.

The 2007 Strategy defines this area as CU14. The Strategy identifies that, in terms of assets being defended, there is little economic justification for maintaining the existing defence line within the frontage. The recent damage and subsequent works to support the sea wall, particularly to the east of the Bay, are indicative of the on-going pressure on these defences. It is recognised that the area is an important visitor and amenity area, and while the current defence line remains in front of the natural line of the shore, it is unlikely that there would be extensive erosion if defences were abandoned or removed.

When considering potential flood risk, this does exist to the rear of the western-most wall. This is also examined, along with the potential wave reflection from this wall and the impact this has on the shape of the main bay. However, the main focus of work would be in examining how the main frontage responds and would respond under different management options

Therefore the 2007 Strategy recommended a Do Nothing policy but recognises that this may need to be modified by land use planning decisions beyond the scope of issues considered by a coastal defence assessment. The first steps towards taking this forward are to examine more accurately what consequences would arise from abandoning the existing defence line and considering further other alternatives.

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6.2 Objectives

- Establish a more robust wave and water level climate, allowing a more detailed analysis of coastal processes and understanding of the critical influences of the Bay's behaviour.
- To provide an improved assessment of the coastal processes for the frontage, defining the natural alignment of the bay and considering the potential erosion extent should defences or sections of defences be abandoned.
- Based on the above, to examine how the Bay may be managed in different ways, considering approaches such as groynes, offshore structures, local control structures or partial abandonment of existing defences; in addition to re-examining potential future requirements should the existing defence be maintained.
- Also based on the above, examine potential flood risk which might arise from loss of defences or the subsequent roll back of the shoreline.
- To provide specific costed outline options, highlighting potential benefits and disadvantages of different approaches for consideration by consultees.

This element of the larger study draws upon information on tide levels and wave climate presented in Appendix A using this to explain how the bay is currently behaving and how it would behave under different approaches to management. The study sets out the baseline information, assesses the longer term risks in terms of flooding and then discusses this in relation to the coastal processes. From this understanding, the study concludes with a detailed discussion of management options.

6.3 Description and Base Line Information

6.3.1 Description of features

Figure 6.2 shows the general arrangement of Pembroke Bay.

To the west of the Bay is a large rock headland. The associated intertidal and sub-tidal rock platform, upon which this headland sits, runs out to the north some 1.5km from the shoreline, representative of much of the nearshore area of the northwest coast of Guernsey. The eastern headland similarly sits upon a wide intertidal rock shoal but extending only some 500m offshore.

The main area of the Bay is formed within these two massive headlands and comprises a broad expanse of intertidal sand over lying the rock platform, with rock outcrops emerging in places over the foreshore. Two quite large areas of rock are exposed in the central section of the Bay (indicted in Figure 6.2 and shown in Plate 6.1).

To the rear of the old military defences is the L'Ancresse Common. This is an area of sand and alluvium infill with similar exposure of rock outcrops. Typical land levels rise from around 6m to 6.5mLOD, behind the defences, to levels in excess of 10mLOD before typically falling away across the Common to the main northern coastal road (Les Clotures) and the low lying land around Vale.

Plate 6.1. The sandy foreshore of Pembroke Bay showing areas of rock outcrop in the distance.

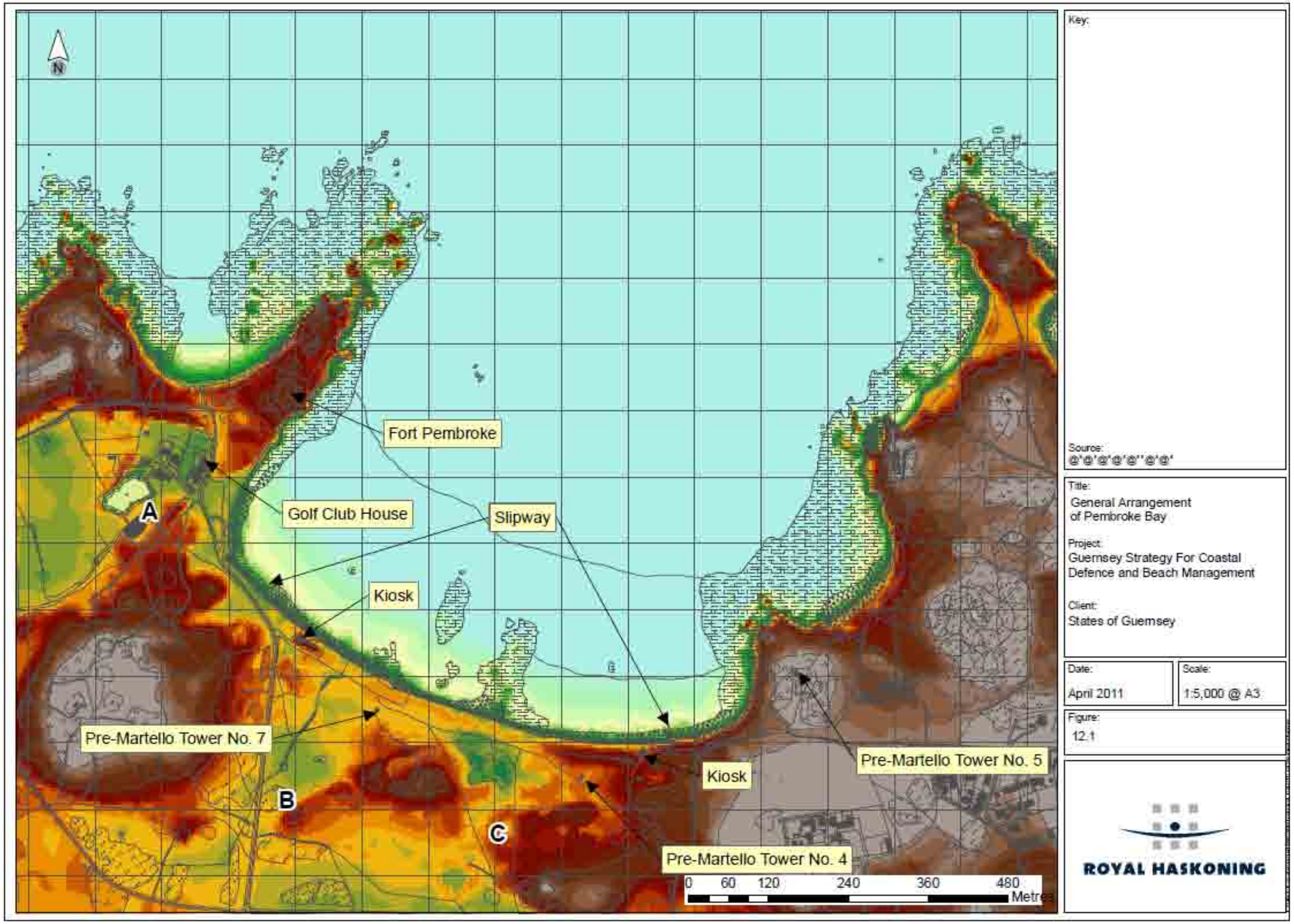


Figure 6.2 General Arrangement of Pembroke Bay showing land features overlying a relief map of the Area.

There are three areas of relatively low lying land running as valleys back from Pembroke Bay, shown on Figure 6.2 as locations “A”, “B” and “C”.

The Golf Club house and various properties are located within valley “A” and the pre-Martello Tower No. 7 lies to the edge of valley “B”. The pre-Martello Tower No. 6 lies to the east of valley “C” (Plate 6.2), with high ground continuing quite close to the shoreline through to pre-Martello Tower No. 5 and the start of the eastern headland.

Plate 6.2. Pre-Martello Tower No. 6, showing also the rising land to the east.

There are two kiosks situated behind the defences. The western one, which has been developed as a restaurant, is located on the relatively low lying land between locations “A” and “B” (Plate 6.3). The eastern kiosk is founded on the higher ground. Both locations have car park facilities, the main car park being at the western end. The main access roads to the Bay run along the western valley “B” and, in the east over the high ground behind the eastern kiosk.

Plate 6.3. Western kiosk and car park.

6.3.2 Foreshore and Defences

The main defence line starts in the west to the southern end of the rock headland. Here there is a short section of quite light rock revetment closing between the main western wall and the hard rock.

The main western wall comprises a mass concrete back wall, with a front berm constructed over a steel sheet pile foundation at the toe of the wall. These sheet piles are exposed to a height of approximately 1m to 1.5m over much of the length of the wall and, although the piles appear not to be corroded through, there is quite severe corrosion (Plate 6.4a). The frontage has a generally low foreshore comprised of small rocks and boulders, devoid of sand apart from at the southern end. There is some evidence that this foreshore has been gradually eroding, with stones bonded to the sheet piles with rust at a level some 150mm above the current level of the general foreshore. The wall has a typical level of 7.2mLOD with the ground levels behind the wall some 1m lower.

Continued loss of beach material (and an associated reduction in the beach level) together with deterioration of the sheet piles is placing the structure at risk. A probable failure mechanism would be the loss of fill material beneath the deck of the front berm with failure initially of this element of the wall. This is likely to cause instability in the main mass concrete wall behind. Failure may be expected over the next 10 to 20 years.

Plate 6.4b shows a photograph (inset), taken prior to construction of the main wall, in comparison with the present situation behind the existing defence. It may be seen that, while the line of defence is set back by potentially some 20m, the backshore did support a narrow dune system, with a sandy upper beach and shingle berm.

At the southern end of the wall the beach turns sandier with a distinct rock berm to the back of the beach. There is a small slipway (Plate 6.4c) acting as the main access point to the beach at the junction between defence lengths DU2 and DU3.

Plate 6.4a. Wall DU2,

Plate 6.4b. Wall DU2, comparison with photograph prior to construction of defence.

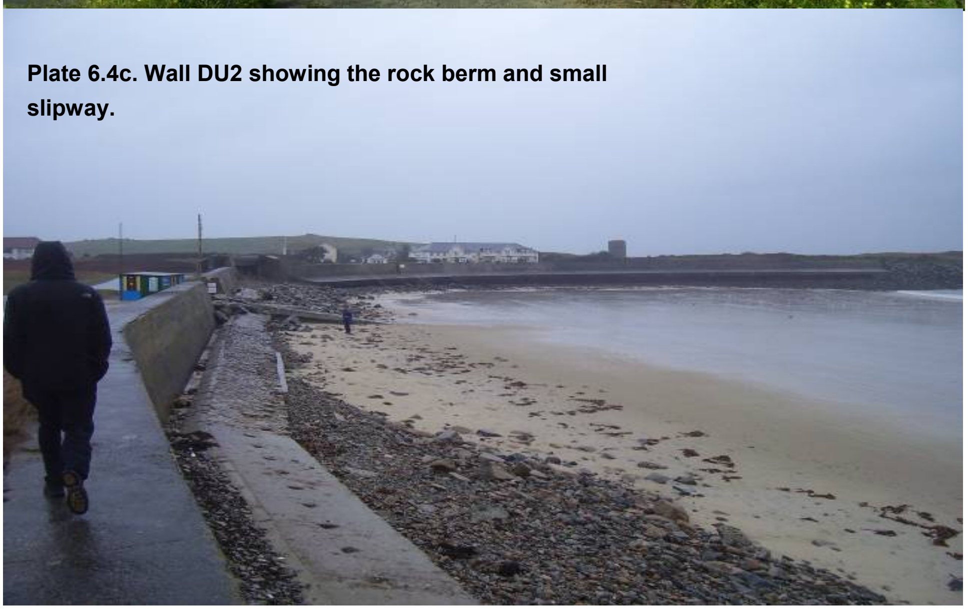


Plate 6.4c. Wall DU2 showing the rock berm and small slipway.

DU3 can be described in several sections but the main defence element is the large mass concrete wall along the backshore.

Over the initial section, running from the slipway in front of the western kiosk, the intertidal sandy beach extends effectively to the toe of the defence, with a small boundary of shingle at the toe. The defence has a substantial concrete toe which has required regular maintenance and repair, especially at the western end where undermining has occurred.

The wall is constructed in straight sections, but each section changes in alignment to reflect the curve of the Bay. However, even small variation in alignment is seen to impact the ability of the beach to maintain a stable shingle toe at the front of the wall. Further east of the kiosk, the shingle bank is wider and is held by the small outcrop of rock at the eastern end of the car park (Plate 6.5).

The wall continues to the east of the rock outcrop but tends to cut across the natural alignment of the Bay (Plate 6.6) such that the shingle bank narrows and disappears. This is compensated for, to some degree, by the second rock outcrop and rock outcropping further down the shoreline allowing sand levels to be retained against the wall.

Plate 6.7 shows the wall beyond the second area of rock, at the point where the wall is taken further forward form the alignment of the bay. There is at this point a marked change in the area behind the wall demonstrating significantly greater overtopping.

This change in alignment also marks a major change in the condition of the wall, with lower beach levels and a severe problem of undermining of the heavy toe. The general condition of the wall to the west is fair, with the main problems occurring at the far western end by the slipway. This section of wall has a typical residual life of some 10 to 20 years with increased maintenance at the western end. To the east of the transition point shown in Plate 6.8, the condition of the wall is poor, with very obvious movement of the wall and regular damage occurring to the toe.

Plate 6.5. Shingle bank east of the western kiosk.

Plate 6.6. Continuation of DU3 between the two main rock outcrops.

Plate 6.7. DU3 looking back towards Pre-Martello Tower No. 7, showing the change in overtopping.

Plate 6.8 shows the variation in the wall over the last four years, despite work to reinforce the toe.

Plate 6.8a and b. Comparison of wall between 2007 and 2011

Plate 6.9. DU4 showing the eastern end of DU3 and the eastern slipway.

DU3 continues in front of the kiosk to end at the slipway at the eastern end of the frontage (Plate 6.9). The poor condition of the wall continues through to the slipway with continued undermining and overtopping along the entire frontage. The level of DU3 is 7.2mLOD

To the east of the slipway is a short section of rock revetment, with a higher shingle beach held by the presence of the slipway.

Figure 6.3 shows an air photograph of the bay showing the back of the Bay in the 1930s. It shows a wide sand shingle beach at the eastern end in relation to the position of the pre-Martello Towers.

Figure 6.3 Pembroke Bay 1930

L'Ancresse and golf course, Guernsey. Aerofilms Series, 43249.

6.3.3 Coastal Conditions

The following information is drawn from the strategic modelling undertaken as part of the overall study and reported in Appendices A, B and C.

Tide and Extreme Water Levels

The Guernsey hydrodynamic model has developed the critical water levels around the coastline (*previous estimates have been taken as equivalent to water levels at St Peter Port). The result was that extreme water levels at Pembroke Bay ranged between 0.22 – 0.23m greater than at St Peter Port (for all return periods). The difference was rounded 0.22m respectively, which were the values associated with longer return period (i.e. more severe) events.

Wave Climate and Exposure

Figure 6.4 illustrates the general offshore wave climate, with the detailed 1:1 year wave conditions modelling for Pembroke Bay shown from a west offshore direction. Results of the wave climate analysis are shown for the two relevant nearshore points, to the west and the northeast of Pembroke Bay. Independent wave roses are shown for swell waves (long period waves originating from outside the area of Guernsey) and for wind generated wave conditions.

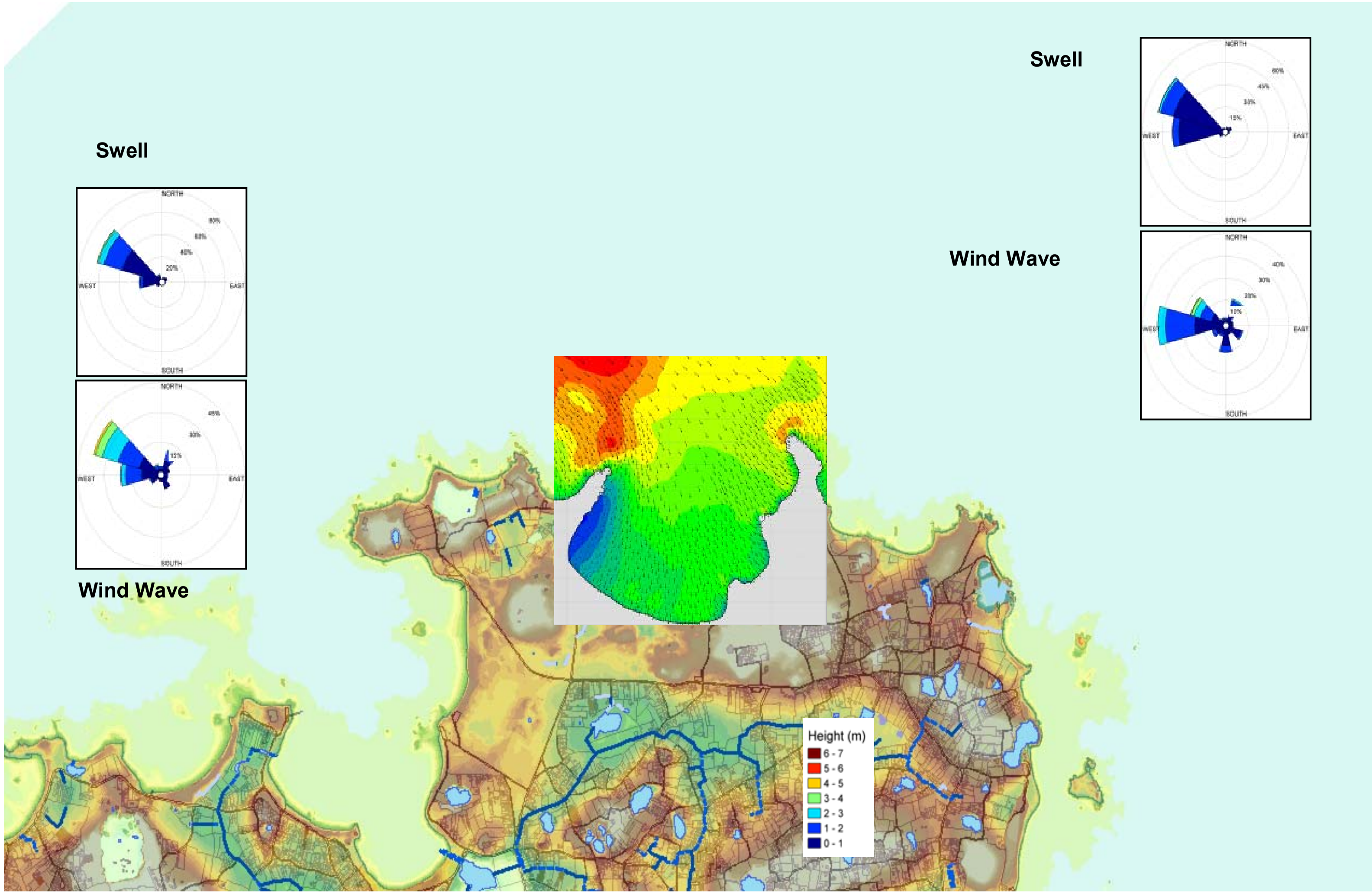


Figure 6.4. Wave Climate at Pembroke Bay.

As with most areas around Guernsey, the wave climate is dominated by waves approaching from the southwest through to northwest. As waves approach Guernsey, they are bent around (refracted) as they encounter the shallow waters surrounding the Island. In this way the dominant wave climate is narrowed to the west to northwest sector. To the northeast of the island the nearshore climate tends to be focussed more from the west.

The plots do however show a significant northeast component of wave energy, which is particularly relevant to exposure within Pembroke Bay.

As waves from a westerly direction approach the shoreline and the Bay, it can be seen that the waves are affected by the shallow waters and shelter extending north from the Pembroke Fort headland, reducing wave heights. Waves are then both refracted and diffracted as they approach and enter the Bay.

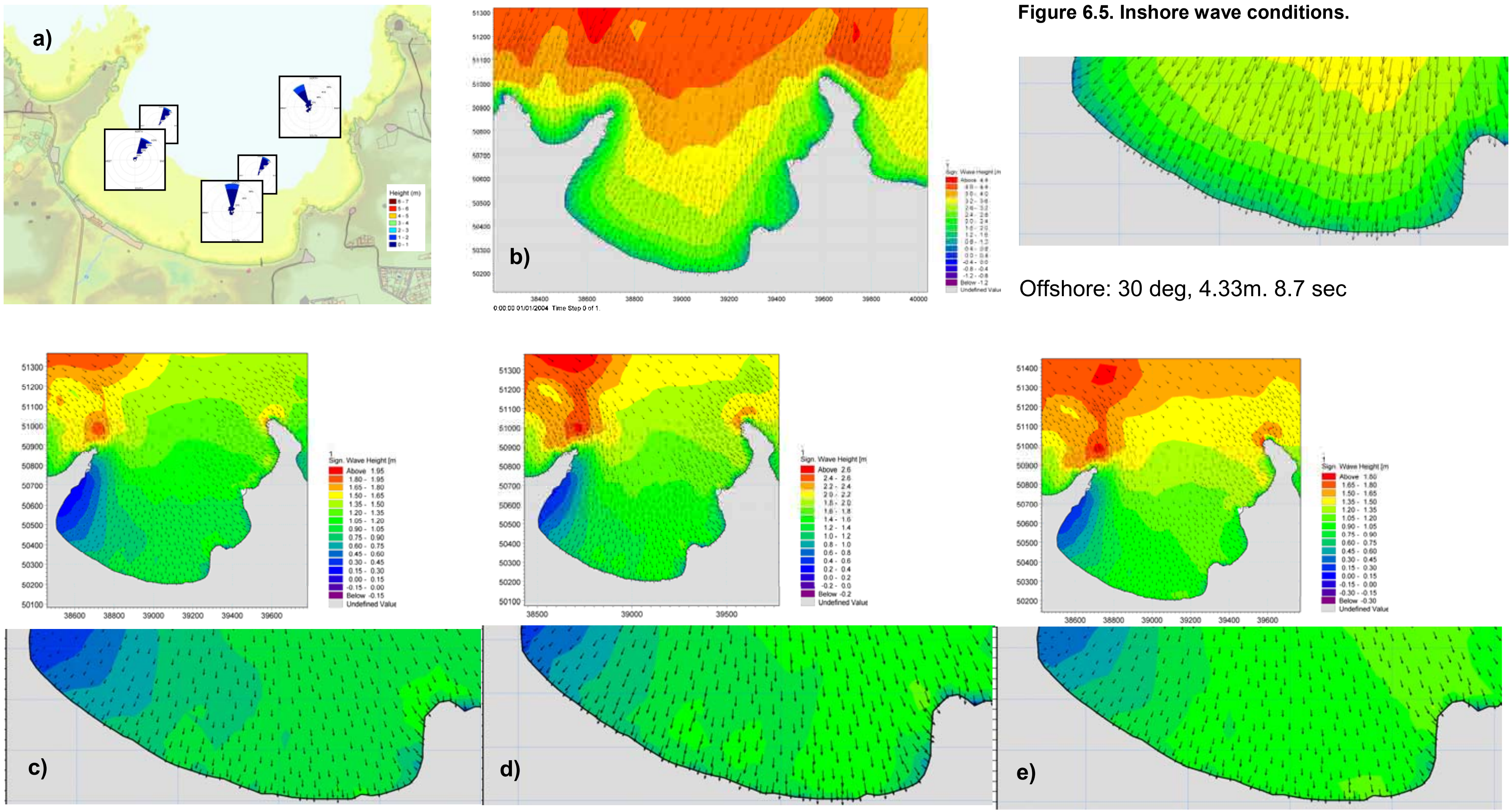
The typical nearshore wave climate and specific wave plots, for both wind wave and swell conditions at two points within the bay are shown in Figure 6.5. An important feature of this comparison is in the directions between swell and wind waves. In the case of the wave point to the west of the Bay, the wind wave and swell are closely aligned, with a very consistent wave direction approaching from the north northeast. Typical average wave heights in this area reach between 1m and 1.5m on a 1:1 yr condition. To the eastern side of the bay, locally generated waves tend to approach the shore from a typical northerly direction but with dominate swell tending the approach the shore more from the north northeast. Typical 1:1 year exposure is slightly greater along this side of the bay with wave heights more consistently of 1.5m to 2m.

Figure 6.5 also shows specific model runs for the main offshore directions: c) 240 deg, d) 270 deg, e) 300 deg. Also shown is the model output for the north northeast offshore condition: b) 30 deg.

Considering the main westerly directions, the most obvious feature of the inshore conditions is the decreasing wave heights from east to west across the bay. Along the most western wall (DU2) wave heights are typically below 0.5m. Despite the greater exposure due to the angle of approach into the bay from the north northwest offshore direction, in terms of frequency, it is waves from the more energetic west offshore direction that gives rise to larger waves within the bay.

The Bay opens to the north northeast and it is from this direction, with waves running directly into the bay that gives rise to the most severe wave attack. On a severe storm, waves just offshore of the defences may reach 2.8 to 3m from this direction. Notably, from this direction, the wave exposure along the western wall increases dramatically, with waves of 2m to 3m running very steeply along the face of this wall.

Figure 6.5. Inshore wave conditions.



Offshore: 30 deg, 4.33m. 8.7 sec

Offshore: 240 deg. 3.6m.10.2 sec.

270 deg. 3.19m. 11.8 sec.

300 deg. 1.9m. 8.8 sec.

In terms of direction, as identified previously, swell waves tend to approach the shore relatively normal to the nearshore contours of the bay. Wind waves tend to be more varied with slight obliquity at the defences. This is highlighted in Table 6.1 showing the variation by offshore direction in relation to the orientation of the defences.

Table 6.1. Wave direction with respect to defence

Offshore direction	***270 deg***	***300 deg***	***30 deg***	**Alignment of wall (deg)**	**Normal to wall (deg)**
Location	**Inshore wave direction**				
West end of DU3	30	30	35	124	34
Central	14	11	23	108	18
East end of DU3	353	354	10	95	5
Angle between wave front and wall (from west + , from east -).					
Offshore dir.	***270 deg***	***300 deg***	***30 deg***		
West end of DU3	+4	+4	-1		
Central	+4	+7	-4		
East end of DU3	+8	+7	-5		

Notes:* difference in wave direction compared to that of the defence is expressed as the angle west (+) and east (-). All other directions are given as whole circle baring.

The greatest obliquity is shown to be at the eastern end of the defence, in front of the kiosk. This is consistent with observations (Plate 6.10). There is a similar interaction at the far western end, with waves potentially forming a bore or Mach Stem effect along DU2 on a north north-east storm and more generally waves working at a steep angle into the corner between DU2 and DU3 by the western slipway. The implications of this wave climate on beach and defence behaviour is discussed below.

Plate 6.10. Oblique wave action.

6.4 Examination of Broad Scale Flood Risk

The Strategy identified potential flood risk areas along this frontage but concluded that this was relatively local to the backshore of the Bay, with the exception of the southwest corner.

Figure 6.6 shows a matrix of still water levels in relation to ground levels for the area, for different return period water levels (highest astronomical tide HAT and 1:100 year extreme water level) and for different sea level rise scenarios (present day, epoch 2 and epoch 3).

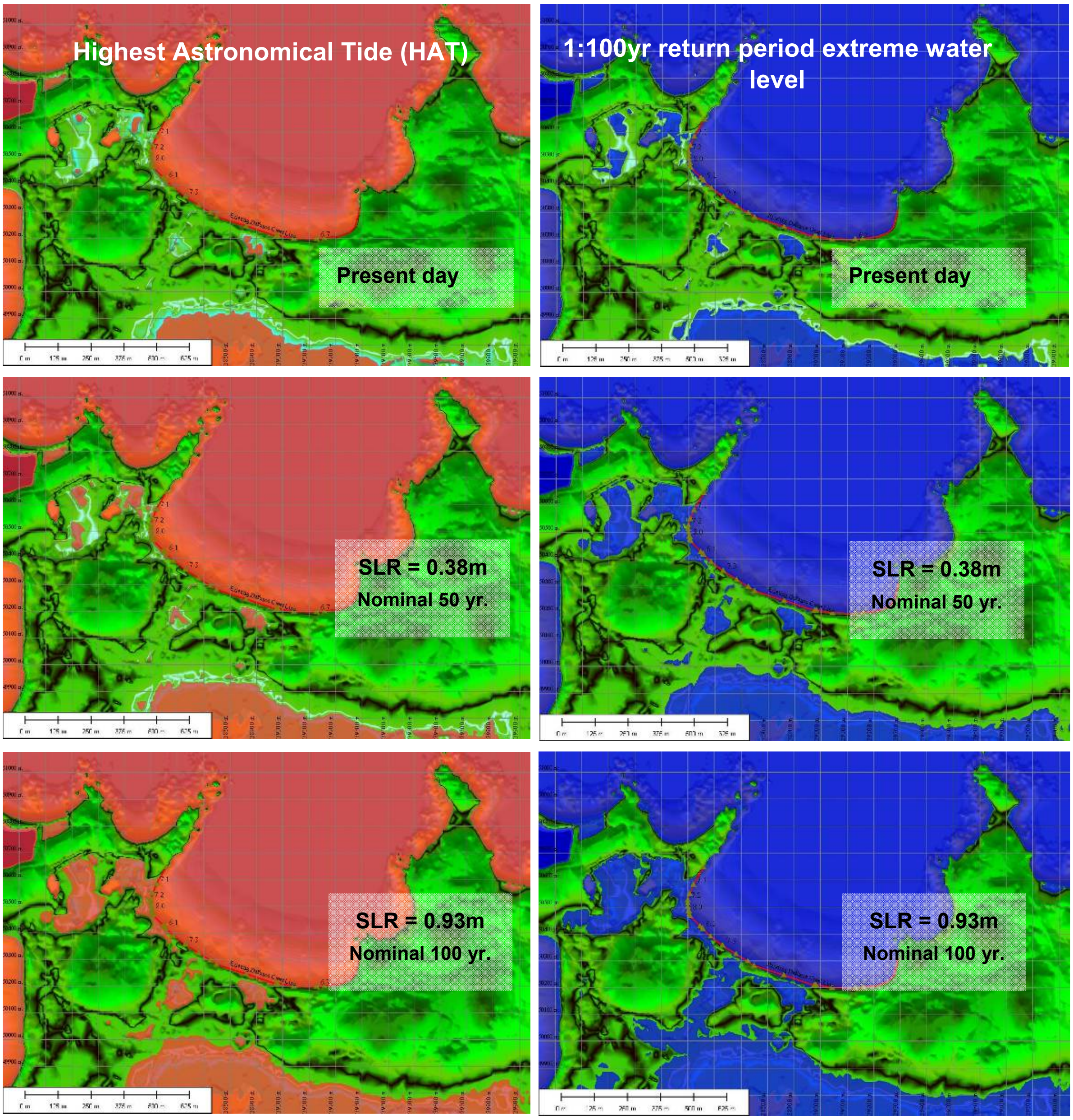


Figure 6.6. Broad scale Flood risk with sea level rise (SLR).

Although there are areas of land in the hinterland below HAT (potential flood areas shown in red), at present the land levels directly to the rear of the bay are such that flooding to the hinterland is unlikely to occur. There is slightly greater risk along the western frontage for the 1:100 year water level, with the sea wall providing the flood defence. However, even here the flood risk is quite local. Overtopping is unlikely to generate sufficient water to cause significant flooding to the local hinterland even along the western frontage.

With 0.38m of sea level rise, for HAT, there would be increased flood risk, similar to that of the 1:100 year extreme at present. In the case of the 1:100 year extreme level in 50 years time, the flood risk to the western frontage increases such that there would be more extensive flooding along the western frontage in the absence of the western walls (DU2 and the western section of DU3). Overtopping is likely to increase and under extreme events this may cause local flooding in the area of the Golf Club house. There might also be flooding to the low lying area, locally to the area referred to as valley “C” in Figure 6.2. The area of the main car park is above the water level but would be subject to increased overtopping. Overtopping would also increase along the eastern section of the bay. This would be due largely to the interaction between the wall and waves.

With sea level rise over the 100 year period, there would be substantially greater flood risk to the western frontage, even on HAT. Although the wall would act as a defence against direct still water flooding, the interaction between waves and the wall would lead to high levels of overtopping. On the 1:100 year water level, severe flooding could occur through the gap by the western slipway but it would only affect the local area. The more significant risk would be through valleys “B” and “C” impacting the main area of L’Ancresse Common and potentially linking through to the large flood zone in the northern section of the island.

While the analysis has shown that there is a major flood risk in the longer term, nominally up to 100 years in the future, management of this risk clearly needs to be taken into account now in developing the present day management approach to the Bay. While the current defence line is capable of providing protection in the longer term, there would be a need to further increase the overall dimensions of the defence to provide long term security. Typically this would require raising defences by as much as twice the level of sea level rise (2m in 100 yrs) to address the risk of overtopping. This would substantially increase the fragility of the defence and increase long term vulnerability of the areas protected which is unlikely to be technically sustainable in the longer term.

6.5 Discussion of Coastal Behaviour

A beach monitoring programme, focused on spring and autumn surveys, has been undertaken by the States of Guernsey over the last decade. This current study has updated the analysis of this information from 2007 through to 2010. A summary of the results of this monitoring is shown in Figure 6.7.

The data collected demonstrates relative change in beach levels between consecutive surveys. The initial plot (upper right hand side of the figure) shows the comparison between autumn 2000 and spring 2001 (winter 2001). The plot immediately below gives the change that occurred over the summer of 2001 and this pattern is continued through the sequence of plots.

The broad pattern of change is for erosion to the back of the beach (shown in red) and deposition lower down the beach (shown in blue). During the summer periods the typical behaviour is for sediment deposition against the back of the beach and lowering of the foreshore lower down the beach. In effect, sediment is being driven up the beach during the summer, during periods when wave conditions tend to be lower and when the process is likely to be driven by the regular longer period swell waves driving across the beach. During the winter, when there is a greater frequency of shorter period, higher waves; the waves impact on the back defence, drawing sediment down the beach.

Beach levels in front of the wall at the eastern end of the frontage are recorded to have varied by as much as 2m over the monitoring period.

This general process provides good evidence that there is no significant overall loss of sediment and that the system, in general, can respond to natural change. The main supply of coarse sediment (shingle and rock) is likely to come from the headlands. This will be relatively low. It is uncertain to what degree sand may be able to be imported from the offshore area.

Quite obviously from the various plots, while there is this large scale long term seasonal behaviour, there is also significant variation at a more local scale. It may be seen that during the winter of 2002, there was quite severe erosion in the area of the upper beach at the western end. This area of erosion tended to fill during the following summer but with some accretion along the lower beach to the east. During the subsequent winter (2003) there was erosion to the eastern section of the bay and some accretion within the western area.

During the winter of 2010, there was severe erosion of the lower beach, particularly to the eastern end, with little benefit gained at the back shore.

These variations can be seen to reflect the differences observed in relation to the wave climate. The process of beach building is well explained by the typical net direction of swell waves entering the bay and working up the normal beach contours. The general sensitivity of wave direction within the bay helps understand the changes in different areas of the beach. This understanding is used in examining in a more detailed manner the different sections of the Bay described below.

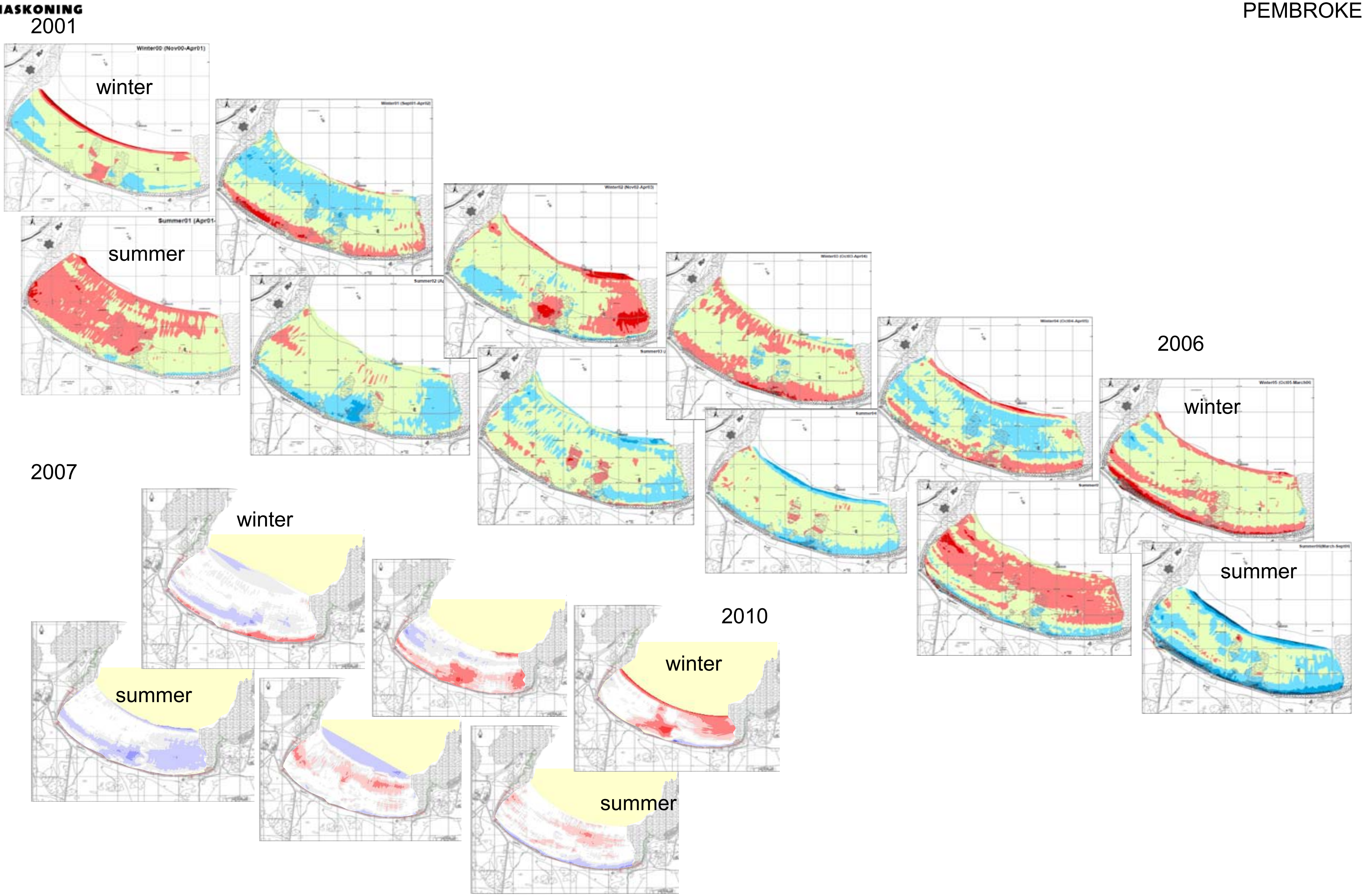


Figure 6.7. Change in beach levels

Western section (DU2)

Generally, beach levels in this area are low, with little sand. Under normal conditions this is a low energy area, wave heights are small in the shelter of the headland and there is limited energy transporting sediment into the area; swell waves tend to move sediment further up the beach. During storms from a north north-east direction, the energy increases significantly, with high waves running along the face of the wall, scouring any sediment and gradually removing the coarser stones littering the area in front of the wall. The modelling suggests that waves will be approaching the wall at a critical angle somewhere between 20 degrees and 40 degrees. The interaction between the incoming wave and the wave being reflected off the wall would generate an edge wave which could be twice the height of the incoming wave. This would be mitigated to some degree where the wave spills onto the lower concrete berm. Even so this action would increase the risk of overtopping and would also pile water up into the corner by the western slipway. The old photograph covering this area (Plate 6.4b) in this area strongly indicates that before the wall was constructed, there was sufficient width to allow waves to spill more gradually along the frontage. However, there is still no evidence of any significant back shore dune and this would be explained by the oblique wave action tending to result in significant long shore drift.

Western Slipway (DU2/DU3)

The increased wave energy in this corner generated along the western wall tends to deposit stone into the corner and this is seen in the relatively high, very coarse, shingle slope. While there is movement of sand into the area, the generally higher energy wave action tends not to allow this sediment to be retained. The area is, under the more westerly wave conditions, still relatively low energy, but wave heights increase as one progresses to the east.

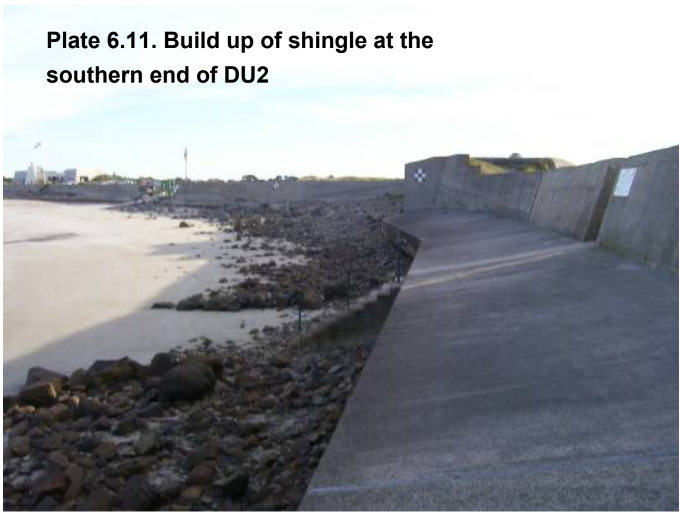


Plate 6.11. Build up of shingle at the southern end of DU2

Considering the angle of wave approach in this corner, it may be appreciated that waves locally at the slipway are significantly out of line with the curve of the wall. Even further east, towards the western kiosk, waves are shown to be approach the wall slightly from the west of north. This contributes to the trend of erosion causing a slow drift to the east.

Records show that the toe to this section of defence is regularly undermined and requires regular repair. This is consistent with the interaction with waves in the area. It may be concluded that the general alignment of the defence is too far forward and would be subjected to regular periods of erosion and a continuing need for management. With sea level rise this problem will become worse. The area is one of the most vulnerable sections of the frontage and is critical to the longer term flood risk management to the local area behind.

Western Kiosk through to the central rock outcrop
The rock to the eastern end of this frontage acts as a groyne. The wave analysis shows how sediment, under all typical westerly storm conditions tend to approach just slightly out of line with the wall and tends, therefore to realign the shingle back beach, exposing the western end of the wall. It would be expected that during storms from the north northeast, waves would be more normal to the wall alignment and would redistribute shingle along the longer length of the wall.

While the rock outcrop maintains control of the drift, the misalignment between the wall and waves is not seen as being too critical. However, there is likely to be a continuing need for repairs as the wall is intermittently exposed. With sea level rise, the pressure on the wall will increase. There would tend to be increased drift to the east and the level of the rock outcrop will be less effective in retaining adequate shingle level in front of the wall.

Central section between the rock outcrops.
The rock outcrop to the western end of this frontage is quite narrow and acts as a cross shore barrier, with little ability to act as a breakwater, modifying the way in which the waves approach the frontage. The larger expanse of rock, further to the east, tends to have a more significant influence on the waves approaching the backshore.

The narrow rock outcrop, therefore, tends to stop shingle moving into this central section, but the rock further down the beach does break wave energy such that finer sand can generally be held over the frontage. This breakwater effect is, however, very sensitive to water level. On lower water levels the rock has a more significant impact on waves, tending to encourage sediment deposition. Under higher water levels, waves can pass over the rock outcrop and can tend to erode sediment against the wall. The monitoring plots for summer 2002 and winter 2003 and winters 2009 and 2010 show this variation quite clearly. Sediment movement against the wall can vary with wave direction and the wall remains too far forward to allow the benefit of the rock outcrops to develop a more stable beach behind.

Eastern section of DU3
This section is under the greatest pressure, with the largest fluctuation of beach levels. There is limited long shore sediment supply to the area, and a strong scouring action due to the oblique wave action in relation to the wall. Sediment supply tends to be from lower down the beach, but the forward position of the wall and the angle of waves along the wall prevents the retention of that sediment. The area is clearly seen from the wave analysis to be the most exposed frontage of the Bay. As a result, the low beach levels, coupled with the higher waves, results in significant over topping. This further acts to destabilise the defences.

Western Slipway.
The slipway acts principally as a groyne. This acts to retain a good shingle upper beach against the higher ground behind. The short section of rock revetment extends slightly forward of where the natural shingle beach would develop.

Overall it may be concluded that over virtually all sections of the Bay, defences are just slightly forward on the natural beach alignment. This is most obvious in the case of DU2 at the western end, where the main issues arise during waves from a north northeast direction and at the eastern end of DU3, where the alignment and forward position of the defence provides no width for development of a natural form of defence. In other areas the defences are generally just slightly out of kilter with wave action, resulting in long term pressure and intermittent vulnerability.

6.5.1 Analysis of the natural form of Pembroke Bay

As a starting point for looking at future management, it is important to consider how the Bay would develop in the absence of defences.

Pembroke Bay is formed as a relative square shape. Although, as discussed above, there are areas where locally long shore sediment drift is an important feature of the backshore, locally, these effects are as a result of the interaction between the dominant wave energy and the defence line. The more natural shape of the Bay would be a shallow sweep in behind the two headlands straightening out over the central section of the bay. This curve would be modified slightly by the natural rock features, and by the relative levels and strength of material backing the bay. The natural bay shape is, therefore going to be very much dictated by the ability of the upper beach to dissipate wave energy approaching quite normal to the general contours of the bay. This can be seen quite graphically in the air photograph from the 1930s, prior to construction of the military defences (Figure 6.8).

Figure 6.8. Pembroke Bay 1930s.

This figure shows a deeper indent to the bay at the eastern end, backed by a solid shingle beach and ridge behind. In the centre of the bay, the influence of the rock outcrop is clearly seen holding the bay slightly forward but with a steeper back cliff behind and by the way in which the rock in the centre of the bay allows the development of a beach and dune ridge over the western side of the bay. Some of the features local to the shore have clearly changed. However based on this photograph together with a historic map from 1938 an approximation can be made of the alignment of the backshore prior to the construction of defences. This is shown in Figure 6.9. The historic map is shown as an insert in the Figure. There appears to have been some form of defence even at this time to the east of the bay in front of a building in this area. This provides a typical baseline for additional analysis of potential erosion that might occur now in the absence of defences in the area.

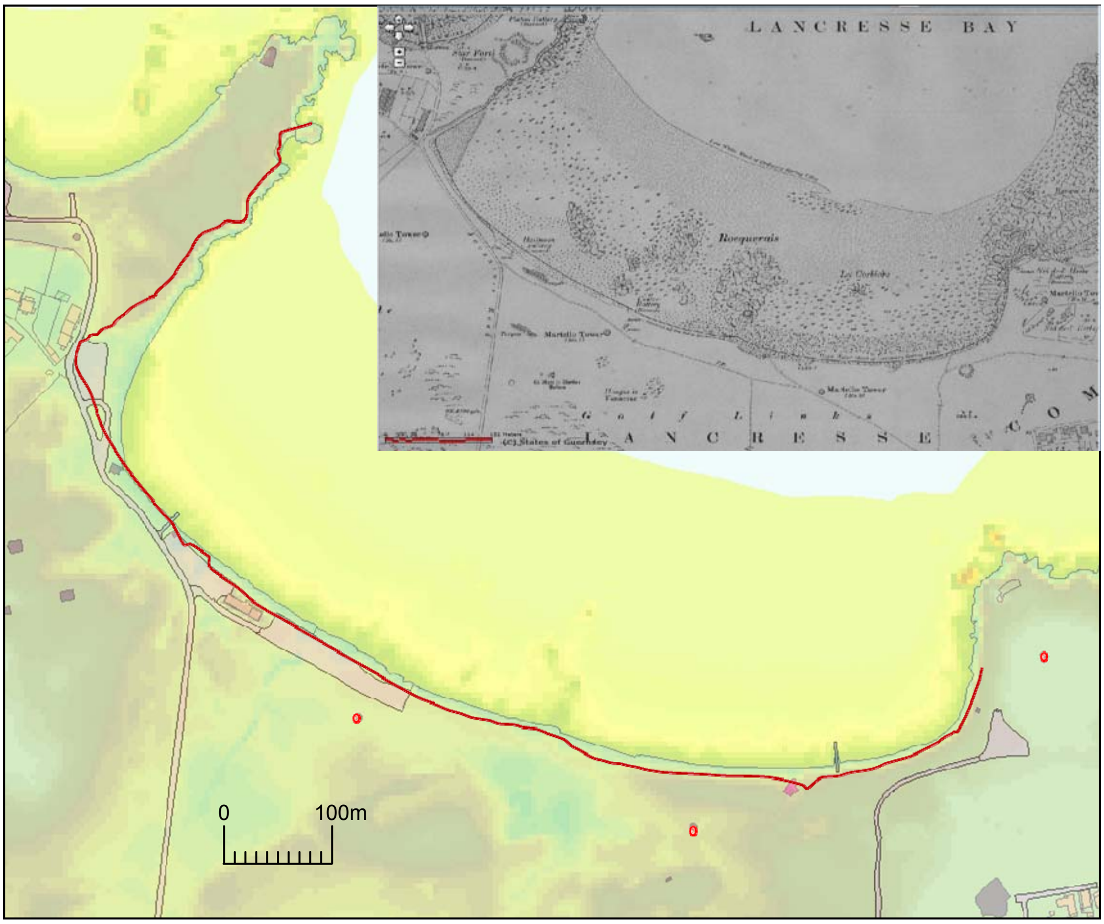


Figure 6.9. Initial Estimate of Erosion based on historic photograph.

The approach taken is based on the beach profile information. Profiles have been taken at various locations around the coastline. This analysis and positions of the profiles are shown in Figure 6.10 (a-i). Profiles a, b and c cover the western half of the Bay, profiles d and e cover the area of the rock outcrop and profiles f, g, h and i cover the east area.

It is immediately apparent that beach levels close to the wall over the western section are generally higher than those to the east. The form of the beach to the western end adopts a more convex shape, consistent with a more nature profile. Further offshore the profiles adopt a more uniform slope.

Figure 6.10 a) profile positions

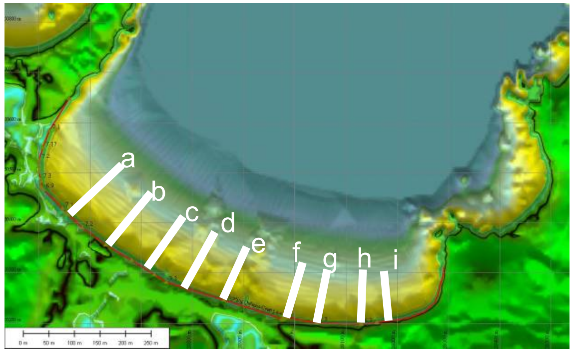


Figure 6.10 b) profiles relative to defence line

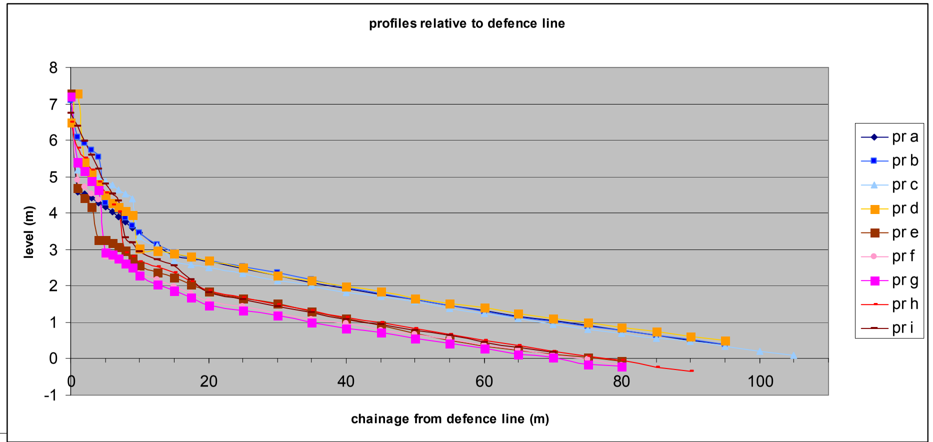


Figure 6.10 c) normalised profiles

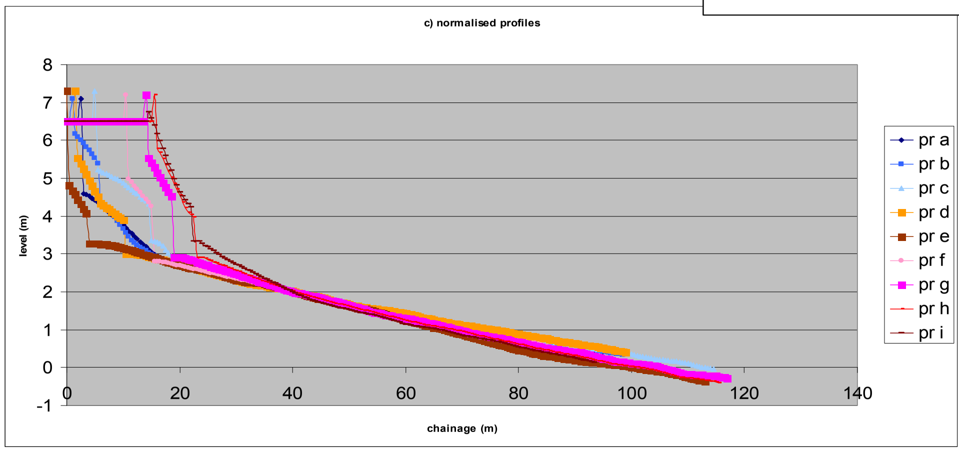


Figure 6.10 d) position of defence in relation to beach.

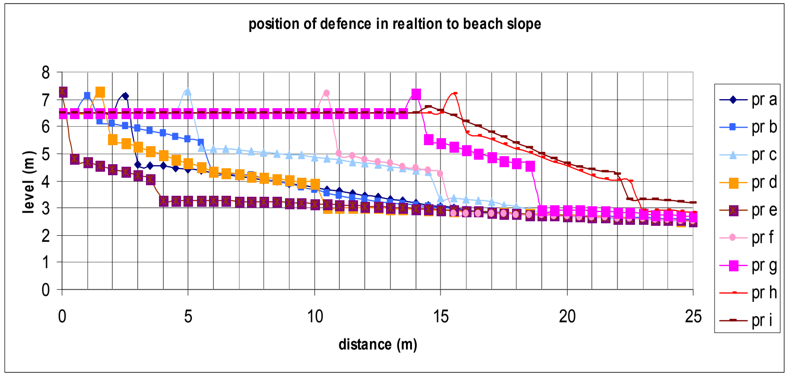


Figure 6.10 Beach profile analysis.

In attempting to assess the degree to which defence may be in advance of the natural line of the beach, it is recognised that the walls themselves would be keeping beach levels artificially low. To correct for this, the profiles have been normalised along their respective chainage at 2mLOD.

This is shown in figure 6.10 c). Profile (e) behind the rock outcrop is taken as the baseline, taking this as a semi natural development of the beach. It can bee seen that profiles (d) and (b) are quite closely aligned to this semi-natural position. Profile (c) is held forward, in effect, by the accumulation of sediment at the crest of the beach, held forward by the groyne effect of the rock.

The profiles most clearly forward are along the eastern end as would be expected. This is shown in more detail in Figure 6.10 d). It is noted that profile (i), at the far eastern end of the frontage is also well forward but this may be explained by the steeper beach as the backshore curves around to the beginning of the rock headland. The beach levels at this location are higher over the upper beach reflecting the stability of the shingle bank created by the shelter of the headland.

Typically from this analysis, it may be seen that in relation to the natural beach form based on profile (e), the western defences are some 5m forward and those to the east some 15m forward.

The profiles give a typical foreshore slope of 1:30 and an upper beach slope of around 1:7.

Given that the level of the wall at profile (e) is still held at the sea wall, it might be anticipated that to complete the profile there would need to be a further set back of some 20m to allow natural development of an upper beach.

Based on this approach the anticipated set back of the shoreline over the frontage may be determined. This is set out in Table 6.3.

Table 6.3. Predicted erosion distances.

Location	Adjustment to alignment (m)	Retreat to a stable crest position (m).
Between western kiosk and slipway	5 -10m	15m – 20m
Western kiosk to rock outcrop.	2m	12m
Central rock section	0	20m
Central rock section to eastern kiosk	15m	35m
East of eastern slipway	5m	5m

As sea level rises, the overall profile of the beach will attempt to adapt. If this change is taken as occurring at MHWS, the probable impact would be for the beach profile to adjust inline with the shallower slope of the foreshore area. Based on this approximation, the horizontal movement of the backshore would be the slope x the rise in sea level. The additional erosion distance is shown in Table 6.4.

Table 6.4. Additional erosion with sea level rise.

Epoch	2011 to 2021	2021 to 2051	2051 to 2101
Additional erosion	4m	9m	15m

The results of this analysis are presented in Figure 6.11 in comparison with the baseline estimate from Figure 6.9.

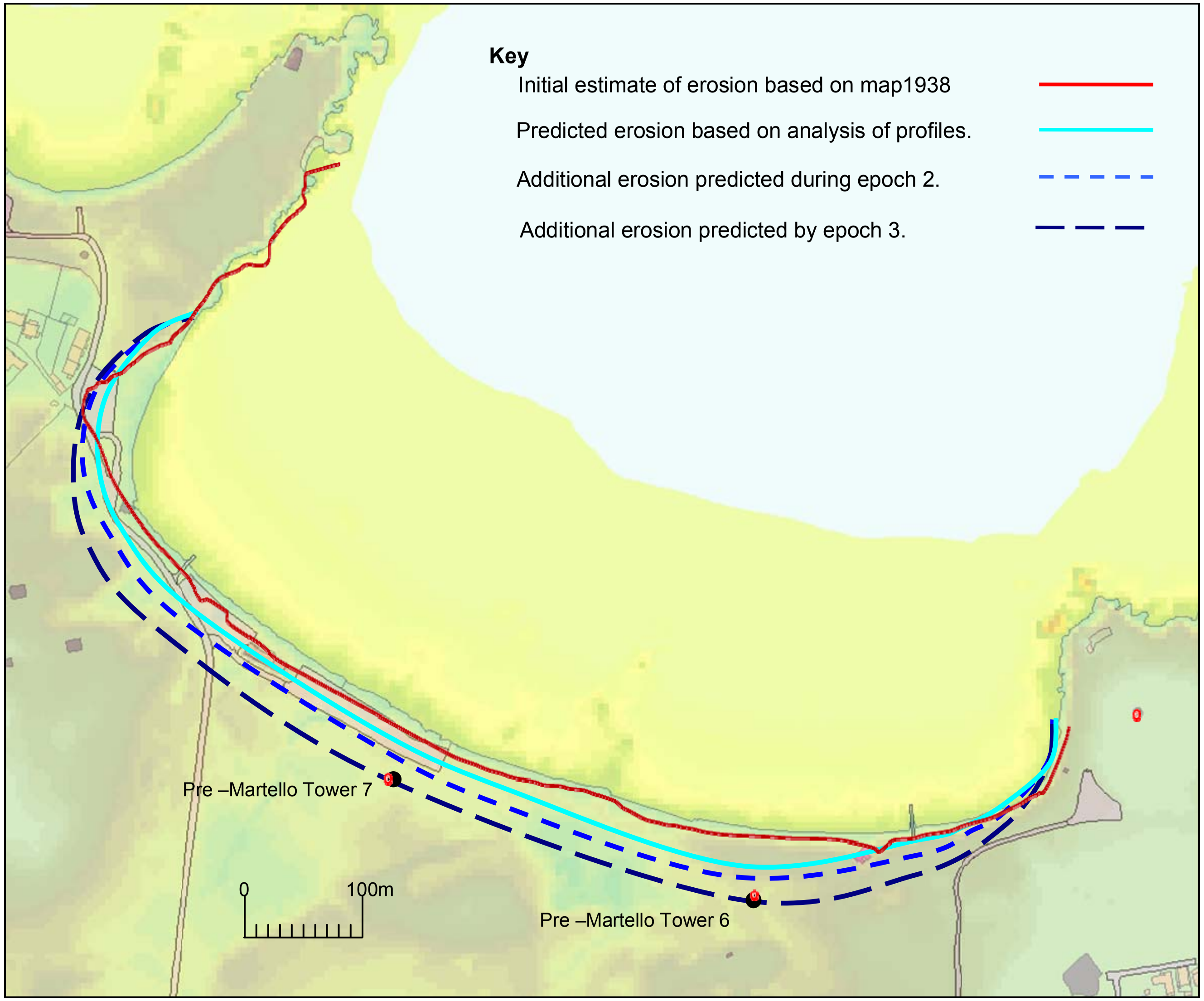
Figure 6.11. Unconstrained erosion lines.

Notes:
Although the erosion lines are based on the beach profiles, it may be seen that the adjustment in alignment brings the shoreline in better alignment with net wave directions.

The erosion lines cut back into higher ground behind the western slipway, in the centre of the bay and at the eastern end. The rate of erosion in these areas may be reduced but would then tend to form a slightly cliffed backshore.

Between these areas of high ground, shingle ridges would tend to form. These ridges would adjust to storm conditions providing a degree of natural flood defence. However, from examination of the old air photograph, there is some indication that shingle has been swept over the ridge at the western end. This would be consistent with the potential sudden change in wave exposure under north northeast storms at this end of the bay.

The road at the western end of the bay could be lost quite rapidly. The two pre-Martello towers could be at risk in the long term depending on the rate of sea level rise.



6.6 Management Approaches

There are a range of potential approaches that could be adopted for management of the frontage. In developing these, it is sensible to bracket the options in looking in outline at two baseline approaches that can be developed; these being the total removal of defences and that of continuing to maintain and improve defences as at present.

6.6.1 Baseline options

Baseline Option 1 - Removal of defences.
The final sub-section 6.5.1 above sets out an option for the natural development of the bay, if all defences were actively removed. It provides one extreme baseline from which to assess the behaviour based on other approaches to management.

The cost of removal of the defences is difficult to define accurately due to the varied nature of individual sections of defence. A typical figure between £2,500 and £3,500 per metre may however be applied; the higher cost being relevant to the larger structures at the eastern end of DU3 and for the western wall (DU2), lengths of approximately 190m and 200m respectively. The remaining central length of 520m is taken at the lower rate. The total cost would be of the order of £2.75M.

Benefit
The primary aim of this approach is to restore the bay to a natural condition allowing the bay to function naturally in the future. The main benefits in such an approach would be in restoring access to the shoreline and to improve that natural amenity of the bay.

The back shore would retreat in line with predicted shape shown in Figure 6.11. Along the main area of the bay there would be an improved sandy beach with areas of sand exposed even at high water. Over much of this area the beach would be backed by a shingle ridge with the potential development of a narrow dune in places. The bay would provide one of the few natural beaches on the island and indeed is one of the few areas where such a beach could be formed. The extent of set back of the shoreline would be limited by the enclosed nature of the bay.

In setting back the line, the natural response at the back shore would be to develop a shingle ridge. This would provide a good level of flood defence across the two main valleys back on to the golf course. This would not provide full protection against extreme water level flooding and further raising of the land may be required to the back of the beach. This could, however, be undertaken through re-landscaping along a line across the common, set well back from the coastal edge. Such works might only be required as sea level rises and as the risk increases in the future, as highlighted in Section 6.4 above. Taking this approach allows a better approach to adaption, creating the opportunity for a more sustainable approach to flood management in the future.

Disbenefit
The obvious and immediate disbenefit would be in the loss of both kiosk and the loss of the main car park areas. There would also be the loss of the amenity area to the western side of the bay and loss of the two slipways. There would also be loss of the heritage value of the military defences.

Associated with the loss of the western sea wall would be the increased risk of flooding to the low lying area to the west. This, initially local flood risk, would develop quite rapidly as the shoreline sets back to its old alignment in this area. The properties would still be well above normal tidal levels and the risk would be from more extreme events. Without action to raise land levels in the area, this flood risk would increase with sea level rise such that properties in the area could be at regular risk from flooding within 50 years and, subject to actual rates of sea level rise would be at risk from normal tidal flooding potentially over a 100 year period.

Flooding is unlikely to impact on the main area of the golf course but in the longer term could affect the Club House.

There would be the loss of the main road behind the beach at the western end of the bay. There is, however, alternative access around the main headland for both properties and to Pembroke Fort.

The two pre-Martello Towers set back behind the bay would not be a risk in the short to medium term. Potentially over the next 100 years, erosion may reach these important historic structures. They would, however, be at the back of a far more stable beach line and local management could sustain these structures without significant interruption to the natural processes.

Baseline Option 2 - Maintain and Improve Existing Defences.

Current practice has been to carry out critical (but reactive) maintenance to sea walls as specific problems have developed. The works undertaken to the sea wall indicate that erosion and undermining around the western slipway and along the eastern section of DU3 has been a long term problem. This has resulted in various works to strengthen the toe of these sections of wall. These toe buttresses have themselves required continuing maintenance to ensure their survival. The most critical area has been at the east end.

At the step in the alignment of the wall, just to the east of the area of rock outcrop, there has been settlement and rotation of the entire wall. This movement is continuing, certainly as a result of undermining, most probably exacerbated by the severe overtopping. No recent works have been undertaken to this section apart from local patching of cracks.

At the eastern end of DU3, in front of the Kiosk, the concrete toe has rotated forward, exposing the toe of the actual wall to undermining with the scouring nature of the waves running along the frontage. The most recent repairs undertaken along this section of the defence were carried out in 2007, infilling the voids and gap between the wall and the concrete toe. In the latest inspection in 2011, it was noted that the toe apron has again moved forward, leaving a weakness to be exploited by the wave action. The recent infilling of the toe over a 65m length will cost in the order of £5k.

The present approach to management can only be considered a stop gap before more major works would be required under this baseline option, to address the underlying problems.

Over the next 5 to 10 years, major works would be required to address these problems and to safeguard other structures around the frontage. The anticipated works are described below.

Plate 6.12. Showing the concrete apron to the eastern wall

On the western wall, in order to secure this structure over the medium term there will be a need to reduce scour and to address the deterioration of the toe piles. The critical element at present is the sheet piles. Under this option the protection would be a rock toe extending up to the concrete apron. This would need to be improved and reinforced with sea level rise, such that the wall would be faced eventually with a larger rock revetment over its full length.

Around the area of the western slipway, there would be on-going maintenance but with the toe being replaced eventually over some 150m of the 310m length by a rock toe. In the future with sea level rise it has been taken that a more substantial rock revetment would be required to safeguard the toe and provide protection against over topping. This work might be delayed over a 50 year period.

The section of wall behind the rock, extending some 210m, is at present less vulnerable to damage. Even so, with continued exposure and sea level rise, a similar approach may have to be taken as described above for the section immediately to the west.

The eastern section of wall through to the eastern slipway is considered to be in the process of failing. Minor works merely patch the problem. The wall is being undermined and each time this occurs, this is likely to increase the overall instability of the wall. In assessing options prior to the repairs undertaken in 2007, the longer term solution of a substantial rock revetment was proposed. This would need to be reinforced over time with sea level rise.

Table 6.5 sets out the anticipated costs associated with on-going maintenance and improvement for each section of the defence. These costs are given as whole costs, not discounted to present day.

Table 6.5. Estimated cost of continued management of existing defences

Location	Length (m)	0 – 25 yrs (£k)	50 - 100 yrs (£k)	Total (£k)
Western Wall DU2	200	200	450	650
Western slipway and Kiosk	310	175	660	835
Wall behind rock outcrop	210	0	450	450
Eastern wall	190	300	300	600
	Total	675	1,710	2,685

Benefit
The primary purpose of this approach is to maintain the existing erosion and flood risk protection provided by the existing defences. This protects the two kiosks and car parks and reduces the flood risk to the local western valley. Over the first epoch use of the coast would continue much as at present, continuing to provide the current amenity value of the area.

There is, however, limited economic benefit derived from this continued defence, as identified by the strategy. The examination of flood risk shows that there is no larger benefit area.

Disbenefit
In fixing the current alignment, there would be an increasing pressure on the defence, with an on-going need for works. Particularly at the eastern end, the critical need to address the failing sections of wall requires significant works and this starts to dictate how this frontage would be managed in the longer term, with further commitment to continued defence.

This applies less immediately over the western and central frontages where works are sustaining the value of the existing defence assets.

Over time, however, there would be a need for greater investment even along these lengths of defence and the overall trend for management would be to encase the whole frontage with rock revetment. This, together with gradually falling beach levels and with sea level rise, less drying upper beach area, would reduce access to and use of the beaches. This would have a significant impact on the amenity value of the area.

As sea level rises, there would be greater reliance on the defence line with greater risk of defences being overtopped and potentially failing.

6.6.2 Alternative Approaches

Both base line options incur significant cost, in the case of Option 1, depending on any phased approach to removal of defences, this cost would occur early on but would notionally reduce to zero into the future. In the case of Option 2, there would still be substantial cost over the initial 20 years, but the main cost would occur over time, with a commitment to increasing cost placed on future generations.

Clearly, to reduce costs there is a further option of walking away from further investment. This Do Nothing approach is considered as Option 3.

Other broad scale approaches were considered within the strategy appraisal; these included major beach recharge and recharge controlled by shore detached breakwaters. These options were costed as £8M and £10M respectively; these options are not considered further.

In considering the two baseline options, there is a clear distinction highlighted between management of the eastern section of defence and that to the west. With respect to the former, there is an urgent need to address the failing walls or to address their failure. In the case of the latter, while there is a continuing problem, this has not reached the same critical condition. There is also seen to be a distinction in use of the two areas, with the western frontage providing protection to the western valley and greater amenity value associated with the car parks, the slightly higher beaches and the road. The natural rock outcrops do also provide a degree of separation in terms of coastal processes. This difference and natural separation may be developed in assessing alternative approaches to management. Based on this, further options considered are:

Option 4 - enhanced protection to the western wall and holding the line over the western section of DU3.

Option 5 - enhanced protection to the western wall and rock groynes along the western section of DU3.

Option 6 - enhanced protection to the western wall and developing shore connected structures to the western section of DU3.

Option 7 - managed realignment along the eastern section of DU3.

These options together with Option 3 – Do Nothing are set out below.

Option 3 – Do Nothing.
Under this approach, rather than positively removing defences, defences would be allowed to fail and the only works undertaken would be to address safety issues.

There would be some cost associated with this option but no significant works would be undertaken.

Major sections of the eastern wall might be expected to fail over the next 5 to 10 years. These defences are large mass concrete structures and would typically fail due to undermining and toppling on to the beach. The structures would be monitored and access behind the structures would be fenced off. As damage was identified there would be a need to close the Kiosk and the small car park.

Once failed, wave action would tend to get behind the walls, undercutting and outflanking adjacent sections of wall. Failed sections of wall would act as low breakwaters, modifying the pattern of erosion behind, tending to form quite steep areas of erosion in the fill material behind. The whole section of wall might be expected to have failed within the next 15 years.

Outflanking would tend to be limited to the west due to the rock outcrop. However, as the general shoreline sets back to the east there may be an increased loss from behind the area of rock, slowly reducing the toe levels at the wall behind.

Potentially the next most vulnerable section of defence would be in the area of the western slipway. In this area failure is more likely to occur quite rapidly during a storm event. Typically this might occur in 10 to 20 years time, with the section of wall becoming increasingly vulnerable to damage as the toe to the wall is lost. There would be continued undermining of the toe and eventual failure of the wall.

Loss of defence in this area could increase the risk of overtopping with wash out and wash over of sediment. As with the eastern section, failure of one section would encourage failure of adjacent sections of wall.

Over the same period of time, there would be continued down cutting of the beach platform in front of the western wall and, as significantly, holes would start to appear in the exposed sheet piling. This would result in voids developing under the concrete berm and this berm may then start to fail. It is uncertain to what degree the concrete berm acts as a support to the main wall behind but it would be expected that there would be undermining and movement of this high retaining structure.

Over the next 20 to 30 years, failure would have occurred along most of the western frontage, with short sections of wall remaining but only acting locally in terms of defence. Over much of the frontage large sections of wall would litter the beach area as the shoreline retreated back. This would not over the long term, necessarily result in a safety risk, although individual sections of failure would need to be assessed with the possible need to remove some sections of failed defence. In particular the western wall is likely to present a problem due to failed and undermined decking and the exposed of the sheet piling.

The cost of managing this is highly uncertain but would be an on-going cost, addressing specific areas. Typically, one might envisage the need to remove the western wall completely, at a cost of some £700,000 some time around year 20 to 30. In other areas, the intent would be only to remove critical sections of failed defence as they posed a risk to safety.

Clearly decisions could be made combining this approach to management with that of actively removing sections of defence as in Option 1. This sub-option may then act to spread cost more effectively and may still allow some planned approach into the future. For example, taking forward this in relation to the eastern frontage, the area where movement of the wall is already happening could, as at present be fenced off, the wall allowed to fail and action then taken merely to tidy up specific areas posing a risk to safety.

In other areas a more structured approach may be required that actually removes the defence.

Benefit
The main benefit would be in reducing costs and spreading the cost of demolition over a longer period of time. Over the longer term there would typically be the same general benefits identified in Option 1.

Disbenefit
There would be no planned programme of change. As such, as defences became more vulnerable there would be the need to evacuate the kiosks in advance of failure and removal of the buildings in a manner determined by the deterioration of the defences.

Over the early years, there would be increased deterioration of the amenity value of the area and areas where visitors to the frontage were excluded from using sections of the sea front and beach.

Over the longer term other damages would occur as identified in Option 1.

The following three options focus on management of the western end of the bay

Option 4 - Enhanced protection to the western wall and holding the line over the western section of DU3.
The main immediate issue along the western frontage results largely from the way in which waves, particularly during significant storms from a north northeast direction, interact with the western wall. This gives rise not only to continued deterioration along the wall itself but also causes erosion at the western slipway and the adjacent wall in this area.

Placing a rock toe along the base of the western wall would to some degree reduce these problems by reducing the level of reflection and reducing the development of the edge wave effect. The cost associated with this is identified as being of the order of £200,000 initially (Option 2).

More effectively, some form of breakwater or groynes could be constructed along the frontage and at the southern end of the western wall. This is shown in a high level outline in Figure 6.12.

These works would not exclude the need for protection along the face of the sheet piles but would significantly improve the effectiveness of this toe while also reducing reflected waves that run along the face of the wall by the western slipway.

The outline estimated cost of the work would be of the order of £600,000 (this includes the cost of the rock toe allowed for in Option 2).

Benefit
The approach outlined above aims to address the exposure created by waves, principally from the north northeast, running along the western wall. This reduces pressure on this wall but would also address some of the scour problems along the

western section of DU3. This would reduce maintenance costs and provide the opportunity for a more stable and higher level of beach in this important amenity area.

If the defences were to be sustained over the full 100 year period, there would still be a need for further work to address the issues of sea level rise. However, the key aspect of moving towards an approach that is addressing the main cause of the problems would be in maintaining the opportunity for adaption in the future.

The approach outlined could be developed further if future defence was deemed sensible. Alternatively, if the initial period of maintaining defences was used to plan an adaptation of use in the area, this approach would still be compatible with any potential future realignment. Future management would be far less driven by the deterioration of existing defences. The main issue would be in addressing the reducing standard of flood defence as sea level rises.

Disbenefit
The main disbenefit is in terms of the additional cost. There would be a cost of the order of £400,000 over and above that estimated in Option 2 during the first 20 years.

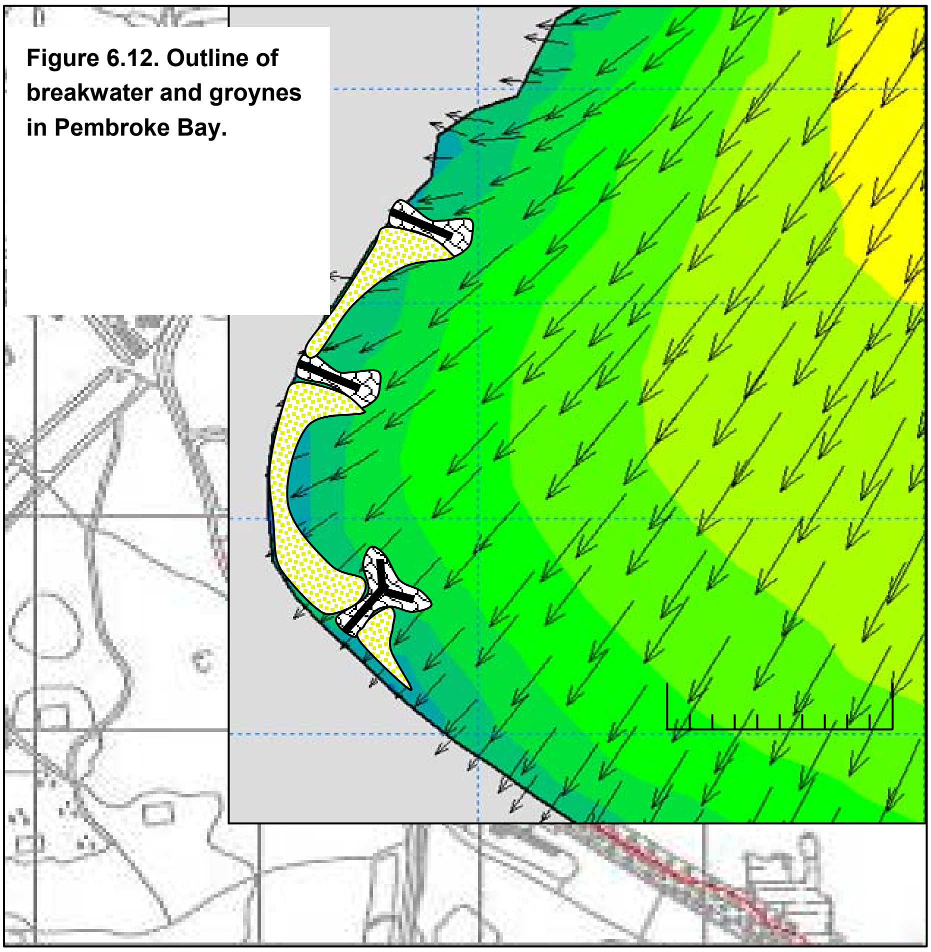


Figure 6.12. Outline of breakwater and groynes in Pembroke Bay.

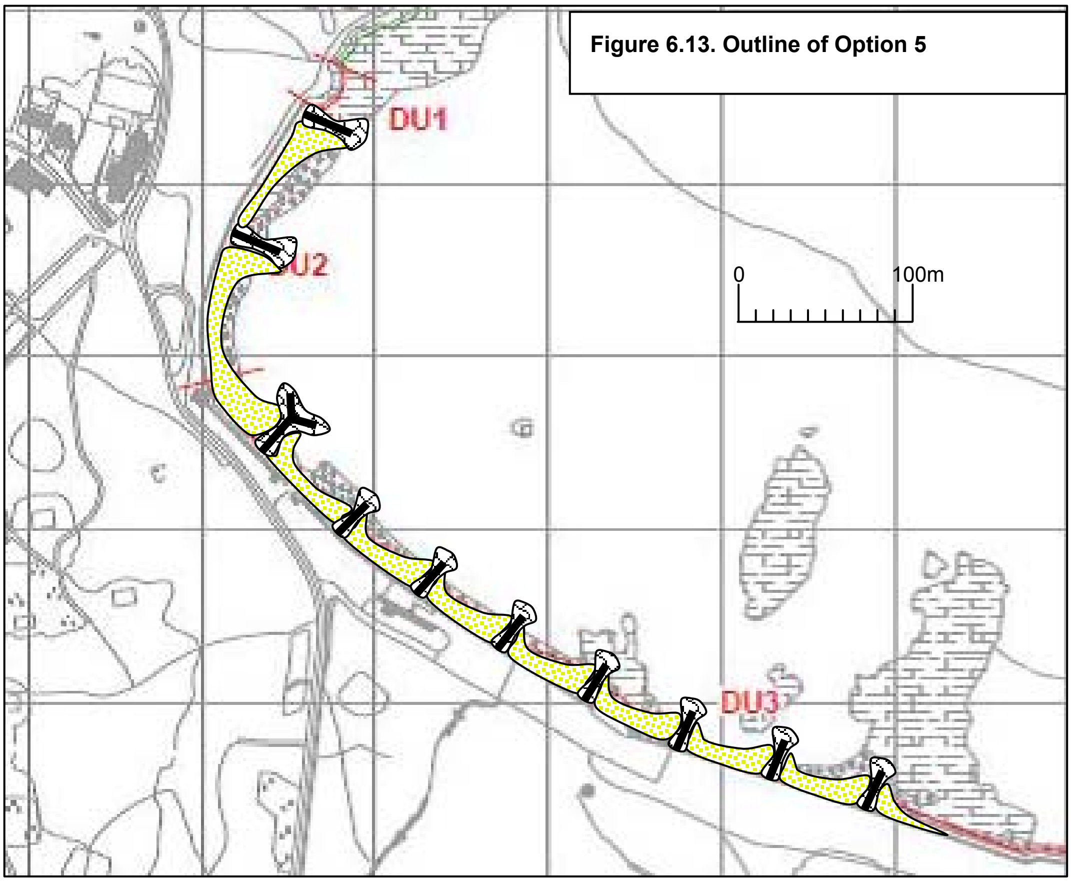
Option 5 - Enhanced protection to the western wall and rock groynes along the western section of DU3.
Option 4 could be developed further in addressing the vulnerability to the western section of DU3. There would not be the same quite the same benefit in that waves

approach this section of defence more normal to the beach crest. However, by placing short rock groynes they would act to improve the stability of the upper beach retaining sediment against the toe of the wall. There would still be the tendency during more severe storms for sediment to be drawn down the beach but not to the same extent as at present.

In outline, groynes would be constructed typically every 50m along the length and would extend possibly some 30m from the face of the wall. This is shown in outline in Figure 6.13.

Typical costs would be of the order of £60,000 per structure. With seven potential, structures covering the frontage, this would amount to an overall cost of the order of £420,000.

Figure 6.13. Outline of Option 5



Benefit

As with Option 4, the benefit accrues from the additional amenity value provided by a more stable upper beach area and in the longer term from taking a more adaptive approach to management. This option would still require further work over the longer term to address the issues of sea level rise.

Disbenefit

There would be a significant cost involved in the work and while there would be a reduction in on-going maintenance to the existing defences, there actual benefit terms of reduced damages to assets would be minimal.

A further risk associated with this approach would be the possible reduction of sediment movement through to the east. This risk appears small given that the rock outcrop in the centre of the Bay already tends to reduce such drift at present.

Option 6 - Enhanced protection to the western wall and developing shore connected structures to the western section of DU3.

This option further develops on the above options providing significantly greater control of the upper beach. In association with works to the western wall, the intent would be to construct a larger structure in the area of the rock outcrop. This in outline is shown in Figure 6.14.

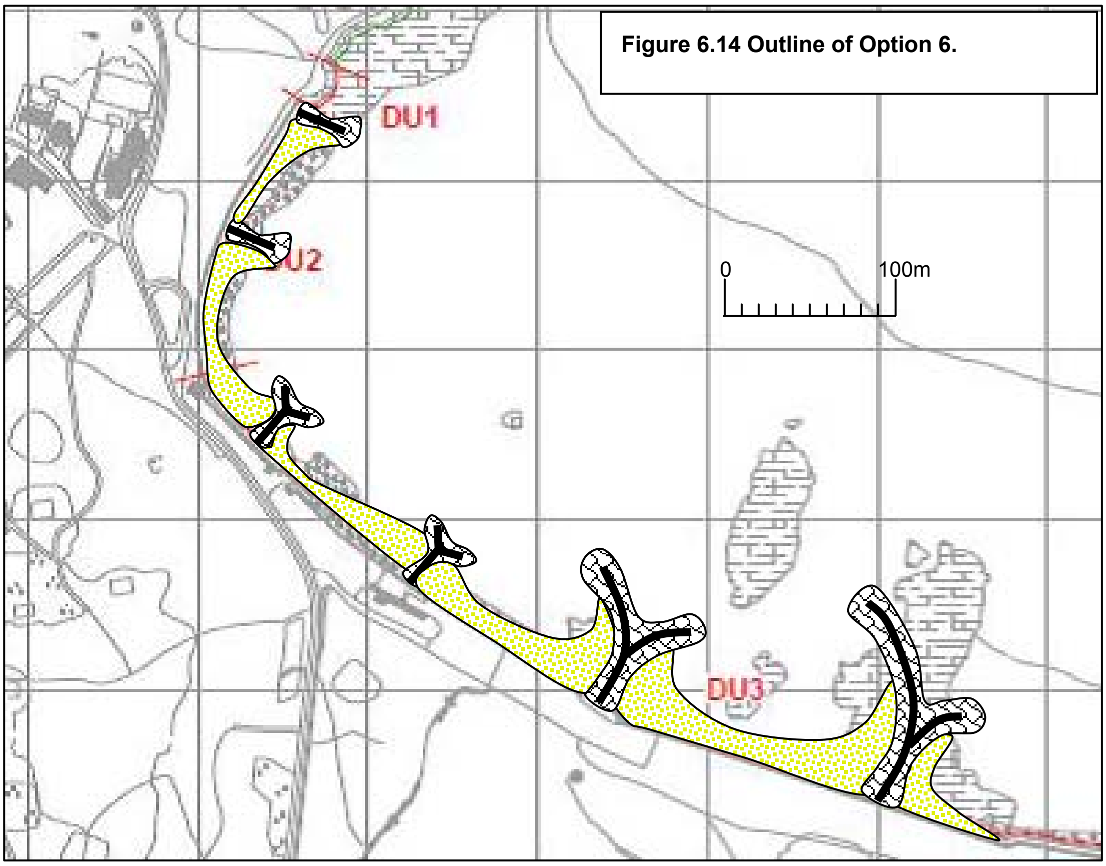


Figure 6.14 Outline of Option 6.

The intent of this approach is, in effect to draw forward the whole shape of the western section of the bay, to create wider beaches and to provide more complete protection to the existing defence line. In doing this there may be the further need to actively recharge the areas between structures so as to avoid material being redistributed as the structures influence the coastal processes. This would need further detailed study and the possible need for physical modelling.

With sea level rise there would be the need for further works but as with other options considered for this area, the approach provides a longer term management that could be adapted to either holding the line or to manage realignment in the future.

Without additional detailed design, there is increased uncertainty within the costs. This uncertainty has however been allowed for. The overall costs, over and above the costs estimated for Option 4, which would form part of the scheme, are in the order of £3M.

Benefit
The option provides a more secure approach to defence over the next 50 years, with the benefit that this approach could be taken forward in a sustainable manner in the future.

There would be improved amenity value in terms of wider beaches and areas of beach that would remain dry over normal tides.

The approach builds on the natural rock base in the centre of the bay reinforcing natural processes.

Disbenefit
There is a significant cost associated with the approach that goes well beyond flood and erosion risk benefits that may be derived from the work.

The large structures would have a significant impact on the landscape of the area, with large structures exposed over much of the tide. These structures could have an impact on the eastern frontage tending to draw sediment into the lee of the most easterly structure and further reducing beach levels to the east.

In summary for the western section of the bay, Table 6.6 sets out the anticipated costs over the initial 50 year period.

Table 6.6. Summary of costs for the western defences

Option	Estimated costs (£k)
1. Removal of defences	2,025
2. Maintain and improve existing	375
4. Enhance protection to DU2 + maintain DU3	775
5. (option 4) + rock groynes	1,020
6. (option 4)+ control structures	3,600

The following discussion focuses on the eastern end of the bay, with two principal options being considered for managed realignment.

Option 7a – Control Structures to Develop the Shoreline in Front of the Existing Defences.

The intent of this option is similar in principle to those considered for the western end of the bay, in that the aim would be to provide a natural defence alignment through use of control structures. This is shown in outline Figure 6.15.

The relatively normal wave approach means that to advance the line of a beach sufficiently to provide continued protection to the toe in this area, structures have to extend a significant distance offshore and provide sufficient shelter to allow waves to spread within the influence of the arms.

Typically, as shown in Figure 6.15, two structures would be used: at the section of wall that is badly undermined as in toppling forward, providing support to this wall, and at the slipway. These structures could be integrated with the various defence approaches taken in managing the western frontage.

The optimum position of the structures would need to be modelled in detail to ensure that a stable beach provides adequate long term protection to the existing defence. There is therefore increased uncertainty in costing this option. There are also significantly greater costs associated with the increased height of the structure in relation to beach levels at the wall. Typical costs are of the order of £1.8M.

Benefit

There is little economic benefit and the prime reason would be to stabilise the existing defence and provide continued protection to the Kiosk. There would be an improved area of beach with some amenity value and the slipway might be improved during the construction of the eastern structure.

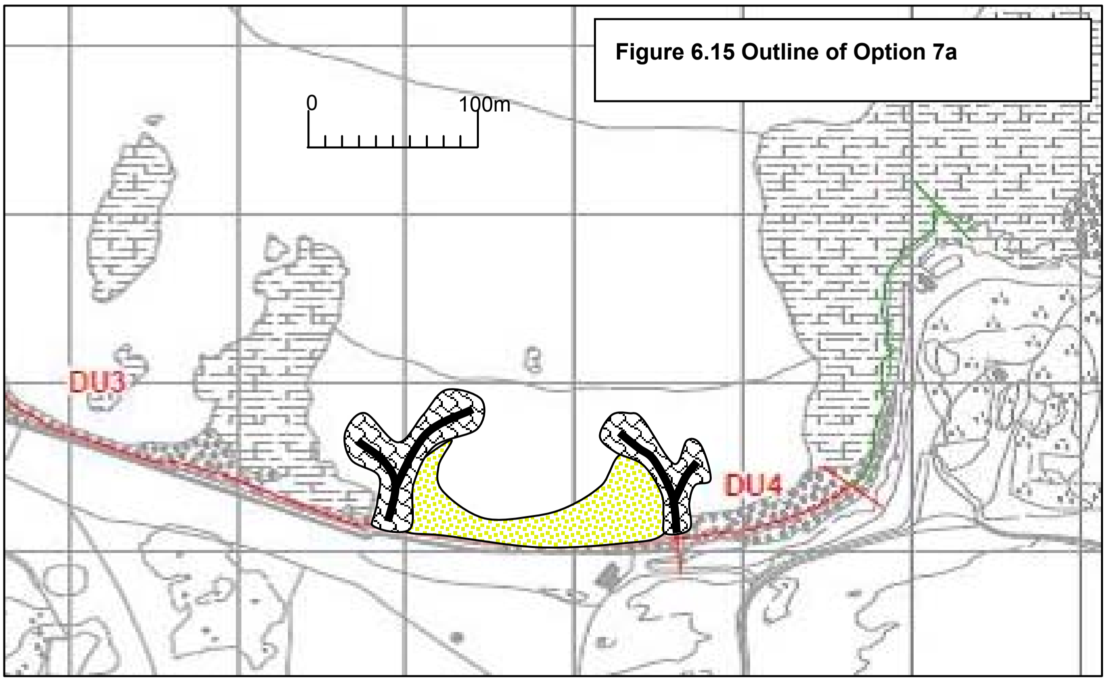


Figure 6.15 Outline of Option 7a

Disbenefit

The main disbenefit would be the extremely high cost associated with the work.

Option 7b – Control Realignment of the Existing Defence.

One of the main difficulties in taking forward the line of defence is the size of structures necessary to create the width needed to hold an adequate beach. The alternative to this would be to allow the existing defence to fail, thus creating width for a beach to develop to the rear of this forward line and to control erosion as this occurs. This is shown in outline in Figure 6.16.

The works would need to be undertaken as part of and as an additional element of works associated with managing the failure or removal of the existing defences. Typically the additional cost would be of the order of £120,000. The aim would be to sustain the defence to the area of the Kiosk and to provide additional protection to the slipway and existing rock revetment

These works might be compatible with works undertaken to the west, allowing controlled adaption of the frontage over the longer term.

Benefits

The main benefits would be in substantially reducing cost of management, maintaining defence to the Kiosk and addressing the immediate problems associated with the wall.

The approach would significantly improve the overall amenity value of the area by allowing development of a semi-natural beach and significantly improving access. The approach would allow sustainable adaptation in the longer term.

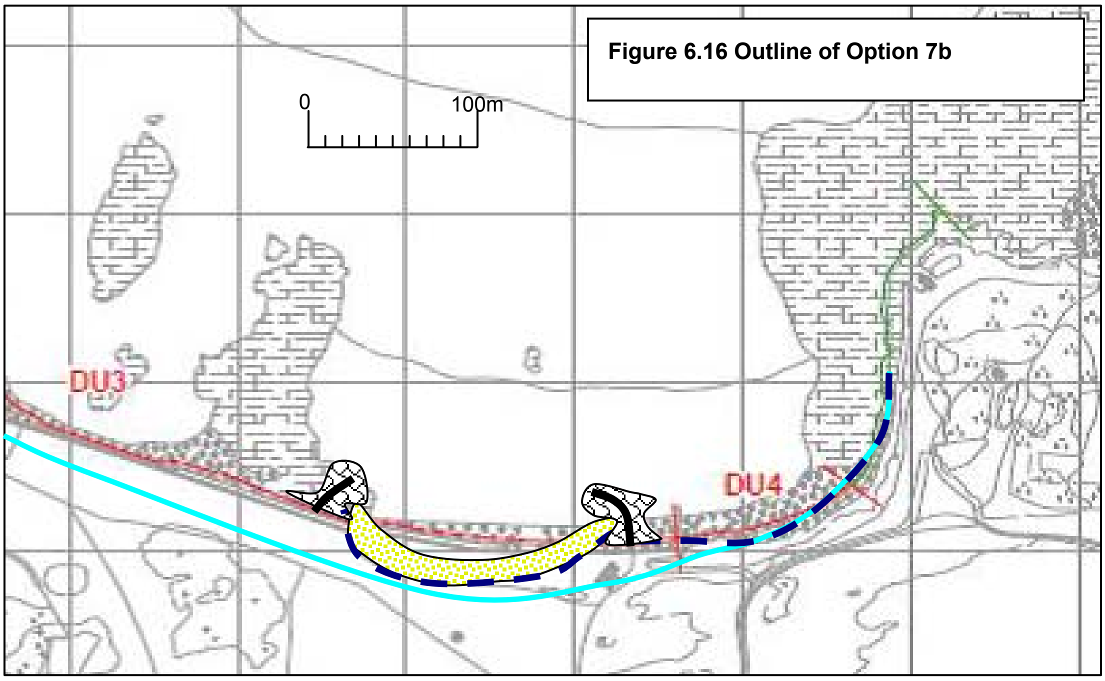


Figure 6.16 Outline of Option 7b

Disbenefit
The main disbenefit would be in some additional cost.

In summary the costs associated with different options for management of the eastern end of the bay are set out in Table 6.7.

Table 6.7. Summary of costs for the eastern defences.

Option	Estimated costs (£k)
1. Removal of defences	665
2. Maintain and improve existing	300
7a. Advancing the defence line	1,800
7b Managed retreat Including removal of defences	660

6.7 Conclusions

The aim of this element of the overall study has been to examine and set out potential options for future management of Pembroke Bay in such a manner as to allow and inform further discussion with interested groups and users of the area. As such no recommendations are made, but conclusions are drawn:

- That the Bay may be considered relatively self-contained with respect to both flood and erosion risk.
- There is a significant local flood risk to the western side of the Bay, but this would develop principally as sea level rises in the future.
- The land behind the Bay is at such a level that it is only into the third epoch (50 to 100 years in the future) that there is likely to be any substantial risk affecting the land to the south of L'Ancresse Common. In addressing this in the future, it is seen as more sustainable to landscape the narrow valleys in such a manner

as to achieve a retired level of protection that will not impact on and force the need for works at the sea front.

With respect to the behaviour of the Bay it has been found from the modelling work, and supporting general observations and monitoring that:

- The Bay is exposed to waves generated from a variety of offshore directions. Although substantially sheltered from the more westerly wave directions, these offshore waves may still generate a significant wave height within the bay, particularly at the eastern end.
- At the western end the wall in this area receives little wave exposure from the offshore westerly waves but is very exposed to waves from a north northeast direction. The orientation of the wall is such that waves run along this west wall and cause scour of the rocky shore platform. These reflected waves also cause significant damage over the western section of the main frontage.
- At the eastern end of the frontage the main problem is the much higher wave exposure from all offshore directions and coupled to this the scouring effect of waves approaching the wall at an angle.

In terms of management, these wave conditions have resulted in on-going damage particularly at the far west end and at the eastern end of the Bay. Various options have been considered to address these issues. These are presented in the report to assist discussion of future management.